ST. FRANCIS RIVER PROJECT

MARKED TREE SIPHON

HYPOTHESIS OF DESIGN

1939

CORPS OF ENGINEERS, U. S. ARMY

U. S. ENGINEER OFFICE   **MEMPHIS, TENN.**

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WAR DEPARTMENT
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HYPOTHESIS OF DESIGN OF SIPHON NEAR
MARKED TREE, ARKANSAS, FOR ST.FRANCIS
RIVER FLOOD CONTROL PROJECT.

I. INTRODUCTION.

1. Authority. - The siphon, which is part of the St. Francis
River Project, will be constructed under the provisions of the Flood
Control Act approved May 15, 1928, as amended by the Act approved
June 15, 1936, and as further amended by the Flood Control Act approved
June 28, 1938.

2. Purpose. - The siphon is to replace a destroyed sluiceway for
the purpose of supplying water for navigation in the 64 miles of the
reach of the St. Francis River from near Marked Tree to Wittsburg,
Arkansas.

3. Location. - The siphon will be located at the foot of St.
Francis Lake in that section of the St. Francis River known as the
Sunk Lands, in Drainage District No. 7, Poinsett County, Arkansas, on
the new left bank project levee of the St. Francis River, near the
navigation lock about 4 miles northwest of Marked Tree, Arkansas, at
approximately river mile 156.

4. General Information. - a. An understanding of the circum-
stances leading to the selection and use of a siphon in connection
with this project is of interest. The Sunk Lands extend along the St.
Francis River from St. Francis Town (Mile 229) to the foot of St.
Francis Lake (Mile 156) near Marked Tree, Arkansas. This reach of the

St. Francis River is low and flat with a channel so poorly defined, and
banks so low, that large areas are submerged during a great part of the
year. St. Francis Lake, which is located at the foot of this reach, is
about five miles wide in its widest part when filled. Before the present
project for the control of St. Francis River floods was authorized,
local interests had constructed levee systems along each bank of the
St. Francis River, in general, between Wappapello (Mile 338) and the
foot of St. Francis Lake (Mile 156). The levees around the lower end
of St. Francis Lake were constructed by Drainage District No. 7 of
Poinsett County. The lake levee system terminates, in the vicinity of
Oak Donnick, in a leveed floodway that diverts the flood waters, which
formerly passed down the St. Francis River by Marked Tree (Mile 150)
and Parkin (Mile 101), and returns those waters into the St. Francis
River, via St. Francis Bay, at a point near Wittsburg, Arkansas, about
15 miles downstream from Parkin. This provides for the protection of
large tracts of land from frequent overflows. The diversion levee
system crosses and dams the St. Francis River about 4 miles upstream
from Marked Tree. Since this reach of the St. Francis River is navi-
gable, Drainage District No. 7 applied for and obtained a permit from
the War Department to construct a navigation lock, sluiceway and that
section of the diversion levee damming the river above Marked Tree.
The permit stipulated, among other things, that the natural flow of the
river up to a volume of 2600 cubic feet per second, shall be allowed to
flow through the sluiceway for the use of navigation. It also stipu-
lates that the height of the sill, across the entrance to the floodway
near Oak Donnick, shall be maintained at such an elevation that no water
can flow down the floodway when the Oak Donnick gage reads less than 10

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feet, equivalent to elevation 210.25 feet above mean gulf level. It is apparent from the foregoing stipulations that the only outlet for waters from St. Francis Lake, below elevation 210 feet above mean gulf level, was by way of the sluiceway located in the levee damming the St. Francis River above Marked Tree.

    <u>b</u>. The levee system, diversion floodway, and navigation structures were completed early in 1926 by Drainage District No. 7. About four years ago the floodway, below Oak Donnick, was widened and extended.

    <u>c</u>. Sometime in 1933 about 40 feet of the outlet end of the sluiceway broke and dropped down at a sharp angle, causing the toe of the levee fill to cave in at this point. Again in October 1936, the safety of the sluiceway and levee connected thereto was endangered by the erosion and resultant caving caused by an eddy set up by the outflow from the sluiceway, which had undermined a row of piling , constructed in 1933, to direct the current away from the bank. Because of this condition it became necessary to stop all outflow through the sluiceway. In May 1938, however, the sluiceway settled and crevassed the adjoining levee. Since the maintenance of these works is essential for flood control and since the works are, as has been mentioned, a part of the St. Francis River flood control project, the crevasse was closed by the Memphis Engineer District Office in 1938. Having in mind the requirements for navigation as expressed in the War Department permit, previously referred to, as well as the general water needs of the reach of the river extending below the levee dam to as far as Wittsburg (71 miles), the Memphis District Office immediately began investigations

and studies for replacing the concrete sluiceway, which had consisted of four barrels, each 8 foot by 6 feet, located under and supporting a portion of the levee crossing above Marked Tree. The investigations, which covered the possibilities of installing a new culvert or a siphon, included detailed office studies, supplemented by field surveys and underground exploration. The underlying strata at the site are fine sands that are inclined to become "quick" when saturated. As a result of these studies, a siphon was selected in lieu of a new culvert. While the culvert could have been satisfactorily replaced by employing cellular sheet piling in the foundation, the cost of constructing an extensive concrete structure of this type would have been about $75,000 more than that of the siphon. The construction of the siphon over the levee offers, moreover, a greater factor of safety from failure due to adverse foundation conditions. Its simplicity of operation and the continuous presence of a nearby lockmaster may also be mentioned as favorable factors with respect to the use of a siphon.

II. PRINCIPAL WORKS.

5. General. - The design of the siphon, as has been indicated, is based on the requirements of the permit with a volume of flow equal to that provided by the sluiceway, which, when in operation, was satisfactory to navigation interests. The principal works will include an excavated inlet channel; a reinforced concrete inlet basin; three 9-foot diameter steel siphons; a reinforced concrete outlet basin; an excavated outlet channel; air and water pumps including piping, valves, and motors; reinforced concrete footings in compact earth fill; a wood trestle across the siphon; a small reinforced concrete wall; and an operating house.

6. Intake Channel. - The intake channel will be about 130 feet long with a bottom width of 64 feet and side slopes of 1 on 2. The 30 feet of the intake channel adjacent to the inlet basin will be riprapped to prevent erosion of the alluvial silt material that is found here.

7. Intake Basin. a. The overall length of the intake basin will be about 45 feet. At its upstream or sill end, the basin will be approximately 68 feet wide; at the inlet lips of the siphon it will be about 60 feet wide. Studies of the mean velocity of approach at this last point indicate that it will be less than three feet per second for the entire range of siphon operation, which will insure tranquil flow into the basin. The elevation of the basin floor, or apron, will be 5 feet below the intake lip of the siphon. This depth will insure a mean velocity, around the siphon lip, of about five feet per second when the siphons are discharging 2,600 cubic feet per second. This depth should, therefore, be adequate.

b. It is to be noted that vortex action may be pronounced at the intake end of the siphon if there is not sufficient water seal depth over the lip of the siphon. This latter depth should be greater than the entrance velocity head, which, in turn, is controlled by the size of the intake basin. Vortex action, in the intake basin, may possibly be caused also by placing the siphons too close together, or by placing the side walls of the basin too close to the siphons. The works are designed to prevent vortex action for any of the reasons stated.

8. The Siphon. a. Each of the three 9-foot diameter siphons will be about 228 feet long. The shape of the siphon will conform, in general, to the cross-section of the levee over which it will be placed. The heel of the outlet leg of each siphon will lie against an ogee section

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of the outlet basin. The invert of the siphon, at the summit, will be placed at the designed flow line of the levee, viz., elevation 226.0 feet above mean gulf level. This is of importance in two respects. The siphon will be so supported that its entire exterior can be inspected for air leaks without unduly encroaching upon the new levee cross section. The pipe will be gateless. If stages, at crest, on the intake side, rise above the invert of the siphon, the air pump, which will be reversible, could be operated as an air compressor in order to prevent flow into the protected area downstream from the siphon, in case such outflow from the siphon was not desired. Open the valves and the siphon acts as a culvert with depth of flow equal to the height of water above the flow line. With valves closed the siphon becomes self-priming.

b. **Siphon Losses.** - In determining the discharge capacity of the siphon, the factors affecting the head required included the following losses.

Friction loss. - In determining the loss of head due to friction, the coefficient of roughness was taken as Manning's $n = 0.012$. This is equivalent to a Hazen-Williams value of $c = 100$ for 20 years service of large pipes, as given in their diagram for the average trend of coefficient for continuous interior steel pipes.

Bend losses. - The head lost due to the several bends was computed from the formula:

$$H = c\sqrt{\frac{\Delta}{90°}} \quad V^2/2g \text{ where}$$

c = a constant assumed = 0.25

Δ = angle of bend in degrees.

$V^2/2g$ = velocity head in pipe.

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The equation is that used by the Metropolitan Water District of Southern California in the design of inverted siphons of large diameters for the Colorado River Aqueduct, and the value of c is based upon a consideration of the increase in the strength of secondary currents developed by bends as the pipe diameter increases.

Expansion loss in outlet transition. - These head losses were determined from values of k in the equation $h = kv^2/2g$, as given in King's Handbook of Hydraulics.

Intake loss. - The intake was considered as sharp edged, and the loss assumed equal to 50 percent of the intake velocity head. In order to decrease the head losses in the siphon and minimize the possible formation of vortices at the inlet, it will be enlarged, by a bell mouthed entrance, to twice the area of the 9-foot diameter section of the siphon, the reduction to the smaller diameter being accomplished in 20 feet. It is believed that this intake size will insure quiet flow conditions at the intake end and that the gradual reduction in diameter will provide an easy change to the higher velocities in the 9-foot diameter section.

Outlet loss. - The outlet end of the siphon will be enlarged to twice the area in the 9-foot diameter section for essentially the same purpose as stated in connection with the intake end of the siphon. The transition, from the enlarged section to the smaller section, will be accomplished in approximately 25 feet. While there is no existing data dealing with the effect of bends upon transitions located immediately below, it has, however, been established experimentally that the head losses resulting from bends occur, not in the bend, but a considerable distance below it. The value of the enlarged transition

section was therefore reduced because of it being directly below the bend. Accordingly, it was assumed that only 3/4 of the exit area would be effective in reducing head losses at the exit, and the head equivalent of the mean velocity at this point was correspondingly estimated. The entire expansion in the transition section will be provided along the horizontal axis of the siphon for the purpose of drowning the hydraulic jump and procuring ebullition throughout the whole area of the stilling basin. It is also to be noted that the outlet leg of each outside siphon is designed with a slight horizontal bend in order to insure the most effective development of the entire cross sectional area at the end sill of the outlet basin, without using too wide a space between siphons or too great a width at the upper end of the outlet basin.

c. Rating Curves. - Calculation of the several losses considered in the preceding paragraphs led to the following expression for use in determining the siphon capacity:

$$\text{Center siphon, } H = 1.508 \; \frac{V^2}{2g}$$

$$\text{Outside siphon, } H = 1.576 \; \frac{V^2}{2g}$$

H is the difference between head and tailwater levels and V is the mean velocity in the 9-foot section of the siphon. The elevation of the upper pool, and corresponding elevation of the lower pool, for discharges through one and three siphon pipes are indicated in Figure I. Figures 3, 4 and 5, are hydrographs representative of extremely wet (1927), normal (1932) and dry (1930) years, respectively, while the sluiceway was in operation. Rating curves for one, two and three siphons are also shown in Figure 9; Figure 10 shows the hydraulic character-

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istics of the siphon pipe, and Figure 2 shows the hydraulic character-
istics of the three siphons for the stated special and rare tailwater
conditions.

d. Siphon Operation. - It is indicated that the siphons will
discharge 2,600 cubic feet per second when full and when the elevation
of the intake pool is about 213.6 feet above mean gulf level, which will
normally occur when the tailwater pool is at or about elevation 209.4.
Duration curve of daily tailwater elevations are shown on Figure 7.
Discharges less than this amount will, of course, be experienced when
the elevation of the intake pool is below 213.6 feet. Low river stages
in the intake pool begin at elevation 210 feet, and stages less than 207
feet provide unfavorable navigation conditions in St. Francis Lake.
Accordingly the operation of the siphons will be regulated so as not to
draw down the intake pool, except under special circumstances, below
elevation 207. The sill of the navigation lock is at elevation 199.3.
The elevation of the lip of the intake end of the siphon is fixed at
203.3, which will insure a suitable depth of seal for vacuum purposes
when the depth of water in the navigation lock is five feet. A satis-
factory seal under extreme low tailwater conditions is also assured,
since the outlet lip of the siphon is fixed at elevation 199.0, which is
about one foot below the general elevation of the river bed below the
outlet channel. Normally the total lift will vary from 22 to 28 feet to
the summit of the siphon or from 17.5 to 23.5 feet to the spring line of
the siphon at that point. When the intake pool is above elevation 213.6
feet, one, two, or all three of the siphons can be operated. Such
operation will provide a wide range in discharge, i. e., from 800 to more

than 2,600 cubic feet per second. The discharge can be regulated by controlling the flow of water in the individual siphons by breaking or providing a vacuum in the summit of the siphon. The vacuum will be produced by a water sealed centrifugal air pump. The vacuum can be broken by opening the 6-inch line through the vacuum pump or the 8-inch valve placed in the summit of the siphon.

e. It is also of importance to note that the siphons become self priming when the velocity of the water passing through them is high, and that this priming action is dissipated when the velocity is low, i.e., air separates from the water and collects in upper part of siphon with accompanying vacuum loss. The velocity of the water in the siphons is a function, of course, of the difference in level between the head and tailwater pools. The greater this difference becomes, the more the velocity increases. Usually, however, when the total lift is greatest, this differential is small and consequently the velocity is comparatively low. While nothing is known of the rate of separation of air from water at low velocities, it is anticipated that with a total lift of 28 feet and a velocity of 8 feet per second that there will be no loss of vacuum.

f. It is of interest to note that when the rate of river inflow into the inlet basin becomes less than the outlet discharge rate of the siphons, the differential between the head and tailwater pools will begin to lessen with accompanying gradual decrease in the velocity and discharge of the siphons. A slow loss in vacuum will accompany the gradual decrease in velocity, since less of the liberated air will be carried out. This loss of vacuum will continue as the outflow from the siphons becomes equal to the inflow. Following this critical period, more

vacuum will be slowly lost and the volume of outflow will become gradually less than that of the inflow. The intake pool will, therefore, begin to rise with accompanying increase in the differential between the head and tailwater pools and decrease in static lift. While the velocity in the siphons will, therefore, begin to increase, it remains to be determined by experiment whether this increase will become sufficient to prevent further loss of vacuum and thereby again stabilize the flow. For this purpose, it is proposed to obtain sufficient data to determine the critical velocity curve at which vacuum is lost at various lifts, differentials, water temperatures, and barometric pressures. With 9-foot diameter siphons it will take many days for this gradual loss in vacuum to take place. It (the vacuum) can be fully restored at any time by running the vacuum pump a short time. This would, of course, cause the siphons to flow full again. The conditions outlined above will occur when the level of the intake pool is below elevation 207, the maximum practical limit of operation for navigation. The proposed experimental data are, therefore, largely for theoretical purposes.

g. Theoretical Lift. - The theoretical possible lift for mean atmospheric pressure at elevation 210.0 feet above mean gulf level is 33.58 feet of water. Allowing for a reduction of 0.60 foot for vapor pressure at 60° and a reduction of 1.61 feet due to reduction of barometer at 28.5 inches of mercury at sea level, the net total theoretical lift available is 31.37 feet. It is apparent that the normal range of total lift (22 to 28 feet) to be anticipated is well within the limit of the net total theoretical lift and, as will be shown subsequently, is

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well within the limit of the air pump capacity. Under the conditions to be anticipated, it is believed that the siphon will flow full when the headwater elevation is less than 207 and that it will continue to do so until the headwater elevation drops to such an elevation that the differential between the pools is so small that the velocity becomes very low.

h. Alternate Siphon Sizes. - The three 9-foot diameter siphons were selected only after a careful consideration of other possible siphon sizes. While the size selected (9-foot) is considered the largest practicable size, siphons of much smaller diameter could, of course, have been used. Less lift when running full and somewhat greater flexibility in operation are the chief advantages in using smaller size siphons. These advantages are, however, offset in that more siphons would be required if a smaller size be used. This, in turn, would require greater widths of the inlet and outlet basins, more footings and supports, more connecting pipes and valves, and greater cost.

9. Outlet Basin. a. The design of the outlet basin is influenced by the fact that the material in the outlet channel (alluvial silt) offers but scant resistance to scouring action. Under the circumstances it is desirable to keep the average velocity over the sill of the outlet basin, which is at the entrance to the outlet channel, as low as 3.5 feet per second. The width of the sill as designed, about 90 feet, insures velocities of this magnitude within the range of discharges anticipated. The depth of the basin will be 10 feet below the end sill elevation of 200. Investigations indicate that this depth will be ample to provide a flow with hydraulic jump drowned under all expected operating

conditions. At the invert line of the outlet leg of the siphon, the concrete section of the outlet basin will be curved in an easy bend to direct the jet, issuing from the siphon, horizontally into the basin without undue shock or turbulence. (See Figure 8).

b. The effective length of the outlet basin will be approximately 55 feet and it will provide a spread of 1 to 6.5 for the jets, with the assumption that the jets will meet each other at approximately midway between the end of the siphon and the sill of the basin. This flare is considered sufficient to insure quiet and well distributed flow over the end sill. As previously pointed out, the divergence of the outside siphons, at the outlet end, aids in making the entire cross section area over the sill effective for quiet flow. The outlet basin and channel are designed to take care of a velocity of as high as 25 feet per second issuing from the siphon. The occurrence of such a velocity appears remote.

c. The elevation of the sill at the end of the outlet basin will be fixed so as to insure a depth of seal of one foot over the outlet end of the siphon. This depth of seal will provide a volume of water sufficient to prime one pipe under the severest operating conditions to be expected. The rate of air exhaust (by air pump) during priming, will be sufficiently slow as to avoid any difficulty from air suck.

d. The downstream portion of the outlet basin will be provided with an upward slope of about 4 to 15. This will be an aid in deflecting any stray high velocity jets slightly upward in passing over the sill. This will tend to "float" such jets downstream before they

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can plunge to the bottom and scour.

    e. The distance between the inlet and outlet basins, 172.3 feet, will be equivalent to somewhat more than ten times the maximum head differential to be expected. This conforms, in general, to the base width of the levee, exclusive of banquettes, at this locality.

    10. Outlet Channel. - The elevation of the bed of the outlet channel will be at elevation 200, which is equivalent to that of the bed of the existing stream where it will be joined by the outlet channel. This elevation (200) also fixes the minimum height of the sill of the outlet basin at the head of this channel. In connection with the plan of the outlet channel, a tailwater rating curve was developed for the most unfavorable condition of stage and flow (see Figure 6). These latter factors are affected by the tributary inflow from Left Hand Chute of Little River and backwater from the Mississippi River. The channel as designed has a base width of 100 feet with side slopes of 1 on 2. Investigations indicate that, under all conditions of siphon operation, the mean velocity in this channel will be less than 3 feet per second, which is the desirable limit. The outlet channel for 50 feet below the outlet basin is riprapped to prevent scour immediately downstream from the end sill and to promote a quiet and well-distributed flow in that portion of the outlet channel not protected by riprap paving.

    11. Air Pump. a. To start the siphons in operation is generally known as priming. It may be accomplished by the removal of air (air pump),thereby permitting the water to flow in at each end to fill the siphon,or it may be accomplished by filling the siphon with water from the summit. The latter method is objectionable in that it would

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necessitate the use of a gate at each end of the siphon, which would require heavy construction and introduce additional head losses.

b. The force which the air pump must overcome is equal to the total lift. The capacity of the air pump, selected in this instance, will be 420 cubic feet of air per minute with a vacuum of 20 inches of mercury, referred to a 30-in. barometer, and a capacity of 100 cubic feet of air per minute with a vacuum of 27 inches of mercury when referred to the same barometric pressure. It is anticipated that this air pump will be of sufficient capacity to meet all practical conditions of operation.

c. It is of interest to point out here that the load on the air exhausting equipment will be materially relieved by the self-priming action of the siphon. The siphon is said to be self-priming whenever the velocity of the water, flowing in the siphon, is sufficient to remove more air at the summit of the siphon than is being released from the water at that point. With sufficient head differential available, the velocity of the water at the summit should, therefore, be sufficient for this self-priming action under conditions of partially filled siphon. To assist this self-priming action the discharge end of the siphon, as has been mentioned, is expanded only in a horizontal direction. This is for the purpose of utilizing the outflow jet in removing air from the siphon.

d. Although the priming time of the air pump is not considered an important factor in the operation of the siphon, it may be mentioned that for a lift of about 26 feet the priming operation will take approximately 100 minutes. For the more usual lifts, the anticipated

time of priming will be less than 100 minutes.

e. The fact that water evaporates with great rapidity as the pressure above its surface decreases (as in the case of obtaining a vacuum), and in so doing creates a gas pressure in the siphon, causes a delay in the time it takes the air pump to provide the desired degree of vacuum.

f. The air pump and the 8-inch vacuum breaking valves will be sealed by means of a centrifugal type water pump capable of pumping 70 gallons of water per minute against a total 55-foot head and self-priming up to 25-foot suction lift. For sealing purposes, the air pump will not provide the desired vacuum when the temperature of the water is above 80° Fahrenheit and it is required to exhaust air at an absolute pressure of 1.04 inches of mercury. For these extreme conditions the water will boil and, therefore, release a large amount of water vapor. Under such circumstances, nearly the entire capacity of the air pump would be required for exhausting water vapor and none would be available for removing the air from the siphon.

g. Since the air pump is designed to handle fluids as well as gases its operation will not be affected by condensation of water vapor in the pump. A condensation tank is, therefore, not necessary.

III. STRUCTURAL DESIGN.

12. Inlet and Outlet Basins. a. The inlet and outlet basins are concrete boxes. The box has its top and one side omitted and rests on a contained sand foundation. A cofferdam is essential to construction and, to reduce the cost, it will be retained to form a cell confining the sand and so renders it a satisfactory foundation and avoids the use of

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wood bearing piles.

b. While the confinement of the sand should make it a satis-
factory material for foundation, a factor of safety against damage from
this source is provided in the design of the floor of the basins. Here
the masonry will be so proportioned and reinforced that it should be
capable of suspension, without rupture, from the sheet piling cell to
which it will be fastened by dowels. If the bearing value of the sheet
piles is 10,000 pounds per linear foot of cell, the cell wall can theo-
retically support the basins without help from the sand foundation.
During construction the hydraulic uplift in the area will be relieved
by pumping, and the elevation of the ground water will be lowered by a
system of well points, uniformly distributed around the area protected
by the cofferdam, which in addition to the inlet and outlet basin areas
will include the rock-paved portions of the channels. Due to flotation
effect the foundation will carry less load after the unwatering pumps
are stopped.

c. To provide for the possibilities of uplift, a value of 12
feet was used in the design of the outlet basin. This should be ample,
as an inspection of the hydrographs of past floods fails to show dif-
ferences of more than 10 or 11 feet in the water surface on opposite
sides of the levee. The floor slab will, therefore, withstand the upward
thrust due to a 12-foot head. The weight of the basin itself, plus the
resistance of the sheet piles, to which it is attached by the dowels,
will also be ample to resist the upward thrust. As an added precaution,
drainage will be provided by means of a gravel blanket covering the
entire area of the outlet basin a minimum thickness of two feet. This

gravel blanket, or filter bed, will be tapped by six 6-inch pipes, each of which is connected to three 8-inch lateral pipes in the filter bed. The drains will also be used for determining the amount of hydrostatic uplift. One drain will also be placed in the inlet basin for this purpose.

d. The side walls of the basins will be reinforced to withstand a wet earth pressure of 60 pounds per square foot to elevation 200. Below elevation 200 the pressure is assumed as 83 pounds per square foot.

e. The inlet basin is not designed for hydraulic uplift. As previously noted, provision for uplift during construction will be relieved by pumping.

13. Footings under the siphons. - The siphon pipes rest on concrete footings placed in the supporting levee and, at the ends, on the back walls of the basins, which are not considered subject to settlement. All of the intermediate supports, however, rest upon a compressible medium. The only way to insure against unequal settlement would appear to be to carry these intermediate footings on piles which would penetrate the sand. The siphon, however, will rest upon rolled fill, compacted at optimum moisture content, which, in turn, will rest upon the foundation it has already occupied as a levee for 14 years. (The existing levee will be removed and replaced as rolled fill to insure against the presence of logs and vegetable matter in the section). The added weight of the slight increase in height of the fill and of the siphon itself should not be great enough to further compress the foundation by any appreciable amount. The fill so constructed should not skrink nor seriously compress under the moderate unit pressures that are

anticipated. To supply, however, a remedy in the event of settlement provision was made for jacking up and correcting the elevation of the bearing plates.

14. **The Siphon Pipe.** a. The maximum distance between the footings, which support the siphon, will be 20 feet. Investigations of the foundation conditions indicate that the 9-foot siphon, when full of water, will be capable of spanning 40 feet without incurring any undue stresses. The siphon pipe will be made of welded steel having a thickness of 3/8 inch in the 9-foot portion of the pipe and 1/2 inch in the outer ends or transition portions of the pipe. Band stiffeners, made of structural steel angles, will reinforce the siphon pipe for the purpose of preventing it from collapsing when under high vacuum. The pipe, as it expands and contracts under temperature changes, can slide on all of its footings, except the one under the summit of the pipe. These footings have sliding plates, held together with bolts in slotted holes.

b. Each pipe will have two man holes, 18 inches in diameter, with frames welded to the siphon shell and designed to be air tight. For checking the velocity of the water, each siphon will have twenty-five 1-inch threaded plugged holes in which Pitot tubes can be inserted. A water gage will be located in each siphon at its summit so as to show the level of the water in the siphon.

15. **Tests.** - Soap. The inlet and outlet basins will be filled with sufficient water to seal each end of the siphon. Air pressure will then be **applied to** the inside of the siphon. All welded places and places where leaks might exist will then be brushed with soap water. Leaks will then be caulked or sealed by adding more welding material. As

the leaks are stopped, beginning at the ends, the water will be raised
in the basins to available maximum elevation, and air pressure increased.

16. <u>Vacuum Test</u>. - The air will be removed from each pipe suffi-
ciently to raise the water within one or two feet of the invert at the
summit of the pipe. It will then be required that no more than about
one foot of this head be lost in 24 hours without any further exhausting
of air from the siphon.

17. <u>Trestle</u>. - A wood trestle, wide enough for one vehicle, will
be provided across the siphons. This structure will be part of the sin-
gle track gravel road system, on the crown of the levee, maintained by
the local drainage district.

18. <u>Wall</u>. - The siphon section of the levee, the top of which will
be 5 feet (elevation 224) below the grade of the adjacent levee, is
brought to grade (elevation 229) by means of a small concrete wall, on
a rectangular location about 16 feet wide and 82 feet long. The wall
and its footing are designed as gravity sections, but will be reinforced
sufficiently to insure against any stresses that may possibly develop.
It is also to be noted that the wall will assist in supporting the
siphons.

19. <u>Trash Rack</u>. - A trash rack consisting of wood piling, five
feet on centers, and two waling pieces, one near the top and one 5 feet
below, will surround the inlet portion of the siphon. A floating boom
will also be provided to prevent trash from entering between the piling.

20. <u>Operating House</u>. - The operating house will contain the air
and water pumps, valves, vacuum gages and tanks.

21. <u>Piping</u>. - The piping will connect the siphons with the exhaust-
ing equipment and water pump in the operating house. The exhausting pipe

will be included in the vacuum test previously outlined. Three 8-inch

diameter compound vacuum and pressure gages, rated from a complete

vacuum to fifty pounds air pressure, respectively, will be installed

on the piping in the operating house.

SIPHON DESIGN PROBLEMS.

In order to enumerate many of the problems considered in the development of this design, they are briefed in the form of questions. Obviously, many of the more familiar ones are omitted.

I. DISCHARGE BASIN.

Will we get a hydraulic jump in discharge basin?

If we get a hydraulic jump, where will it occur?

Will we get a separate hydraulic jump for each pipe?

Will we get any jet action which will extend beyond basin, from high velocities in siphon tubes?

With velocity of 30 feet per second in 9-foot section of pipe:

   a. What velocity with three pipes running full do you expect
      across lip of basin?
   b. What velocity do you expect at lip of discharge pipe?
   c. Will you get higher velocity across lip of basin with
      one or three pipes operating?

Would the discharge basin be satisfactory if 20 feet shorter in length?

Is the width of basin correct at rear?

Would there be any advantage in further widening discharge end of pipe?

Would shape of discharge end of pipe similar to intake be a better design?

Depth below intake is 5 foot and below discharge is 9 feet:
   a. Do we need a differential in depth between the two?
   b. Is the 9-foot depth correct for discharge basin?
   c. What depth is correct?

Will we get any cavitation in discharge basin?

Do we need any baffle blocks in discharge basin?

With velocity of 12 feet per second in 9-foot section of pipe, will we get ebullition or turbulence in discharge basin?

- 1 -

With velocity of 30 feet per second in 9-foot section of pipe:
  a. Will we get violent turbulence?
  b. Will we get a back roller?
  c. Will we get vibration in pipe?
  d. Will we get tremor of concrete in basin?

Would a rectangular basin be a better design?

Would a vertical pipe give a better discharge condition?

If the bottom of discharge basin were paved with hand placed riprap:
  a. Would any of it be moved with velocity of 12 feet per
      second in 9-foot section of pipe?
  b. Would any remain undisturbed?
  c. Where?
  d. With velocity of 30 feet per second in 9-foot section of
      pipe would any remain undisturbed?

Will concrete under discharge pipe wear rapidly?

If sand is carried in water:
  a. Will any settle in discharge basin?
  b. Where?

Will the water in discharge basin surge?

What velocity across lip of discharge basin will disturb hand-
placed riprap in channel?

Will we get eddy action in channel below basin lip at the higher
velocities in pipe?

Will channel riprap be disturbed at 30 feet per second in 9-foot
pipe?

With water surface 213 feet in lake and water surface 208 feet in
discharge basin will you have uplift pressure under discharge basin with
no pressure relief pipes installed?

If lake level increases to 226 feet, will you increase or develop
uplift under discharge basin? How much?

If pressure release pipes are installed under basin, will they be
effective?

If well points are left in place in discharge basin, can they be
used as pressure release pipes?
  a. Is there any advantage in pulling the points and grouting
      the holes?
  b. What advantage?
  c. If left ungrouted will water flow from them into basin
      under any condition of operation?
  d. What condition?

If a well point is pulled and replaced with open end pipe, would you have a flow of sand under any operation condition?

The foundation material is very fine, clean sand. Would it improve the design to drive compaction piling within the area contained by the sheet piling?

Are foundation piling necessary?

The basin walls are attached to the sheet piling by welding reinforcing bars to the piling. Will this impose any of the load of the structure onto the sheet piling during construction while the basin is pumped out?

When water is let into the basin will the load on the piling increase or decrease with pressure release pipes installed?
  a. Is the sheet piling under the discharge basin any advantage as a cutoff wall against piping?
  b. Against seepage?
  c. Against underground water flow?

If the discharge basin was not anchored to sheet piling would you get any settlement? How much?

The sheet piling inclosing the discharge basin acts as a cofferdam during construction. Being left in place, if it were not fastened to concrete walls, would the weight of the concrete squeeze the sand up between the concrete walls and the sheet piling?

What is probable life of the sheet piling?

After the sheet piling rusts out will the basin settle?

Twenty foot of piling below the basin would have been more economical than thirty feet:
  a. Would 20 feet be satisfactory?
  b. Would 10 feet be satisfactory?

After the sheet piling rusts out, will we get piping?

Should part of the sheet piling be renewed before it rusts out completely? What part?

The rear wall approximates an ogee section of a spillway. It is also the last footing for support of the pipes. After designing for carrying this load and for retaining the earth fill, is it necessary to provide mass concrete to withstand vibration in the siphon pipes?

If there is no vibration in the pipes, is there any other factor which makes mass concrete a good design? If so, name the factor.

II.   TUBES.

The cross-section of the intake of the bell is twice that of the 9-foot pipe. Will the entrance velocity be one-half of the velocity in the 9-foot section?

What, if any, other shape of entrance design would be more satisfactory?

The critical volume of flow for design purposes is 2600 cubic feet per second at approximately lake level of 213 feet. Is the elevation of intake lip of the pipe at 5 feet above floor of basin sufficient to give adequate entrance area?

Will the flow lines be stream-lined at entrance to the pipe or will we have turbulence?

Will we have vortex action at each pipe with flow of 2600 cubic feet per second?

Will the flue or duct of vortex extend below intake lip?

Will enough air be admitted by the vortex duct to cause gulping or loss of vacuum?

If the vortex action is severe, can it be stopped by floating a raft over the vortex area?

Will the vortex action decrease rapidly as the lake level is lowered below 213 feet?

Is it correct to state that the siphonic head is 33.58 feet less losses due to water vapor, and normal barometer changes?

This gives us approximately 31.37 feet for theoretical lift. Can we obtain this lift in a practical design?

Under the limitations of this problem can we obtain this lift?

When we have a high lift and a low velocity through a siphon will air separate from the water and collect in the crown?

We know from our automatic siphons that "with a low lift and a high velocity air is trapped and expelled from the siphon". With the design of this siphon will it, after priming, maintain its vacuum with a low lift and a high velocity?

After becoming partially primed, will it automatically complete its priming if vacuum pump is stopped?

If this siphon at 28 feet lift and velocity of about 8 feet per second starts collecting air at the crown, will it be a period of hours or days before flow in the pipe ceases altogether?

We have been taking the maximum lift at the top of the pipe for the vacuum lift. When should the center of the pipe be used in our calculations?

Is the volume of flow in the siphon controlled by the differential between the pools or by the vacuum lift?

When is the volume of flow affected by the vacuum lift?

How are friction losses affected by vacuum vs. pressure?

Granted that a siphon flowing partially full has sufficient velocity to become automatically priming, can you, by letting air into the siphon through pet cocks, regulate the volume of flow?

Can you prime another siphon by operating valves between them after one has become automatically self-priming?

Would there be an advantage or disadvantage in using an elliptical tube?

What chief objection would you have to the use of concrete tubes?

What kind of steel would you use in the siphon tubes - soft or hard? High carbon or low carbon? Rust resistant or boiler plate?

Would you reinforce the tubes with angles, bands or wide rings?

Would the tubes need as much reinforcing near the lips as at the invert?

Should the supporting saddles be designed for the tube to expand and contract by sliding through same or should the sliding occur on a base plate?

How much bearing surface should the tube have on the saddle?

Should the saddles be placed at the maximum spanning distance for the tube as a continuous beam?

Would this maximum distance overload the footings and cause settlement?

What will happen if we get uneven settlement of the footings?

Should we design for easy realignment, if we got settlement?

Have we any flexibility in this siphon tube?

Should the tube be anchored to one footing? Which footing?

Can this tube leak and saturate the levee?

The cost of a power line from the site to a main line was estimated at $7,000.00. The old sluiceway had a turbine and generator. The lock is at present operated by a gas unit. It is estimated that a turbine and generator can be installed for $6,000.00. Can this unit be operated in a 30-inch Y tube taken off a siphon at the elbow above the discharge basin?

III. INTAKE BASIN.

What is the maximum velocity advisable over the lip of intake basin?

Will the entrance losses increase very rapidly as the differential between the pools increases due to the limited depth under the lip of siphon?

Is it necessary for the entrance of inlet basin to be as wide as discharge end of outlet basin?

Is it necessary to design the inlet basin against vibration as strongly as the discharge basin?

Should the floor of inlet basin be of lighter construction than the floor of outlet basin?

Are pressure release pipes desirable under floor of inlet basin?

Under what conditions will you get uplift pressures under floor of inlet basin? What will this pressure be?

Would an inlet basin of sheet piling and riprap be satisfactory?

IV. LEVEE, WALL AND TUBES.

The distance from basin to basin is 172' 9-1/2". The levee is a turnover in place on the old foundation. The gumbo clay is compacted by tractors in 6-inch layers. The river bed is elevation 200 feet; natural land surface about elevation 212 feet; flow line 226 feet; (Note:- Maximum lake level expected is 226 feet). Is a sheet pile cutoff wall under the sea wall desirable or necessary?

If the cutoff wall of sheet piling is used, can it also be used for bearing piles for sea wall?

If no cutoff wall is used, then does it become necessary to put in bearing piles under the sea wall?

Will the levee foundation settle? Will we get further compaction in the levee? How much estimated?

It is assumed that, with siphon tubes lowered 3 feet at a lake level of 226 feet, water would flow through tubes 3 feet deep with air valves open. Is this assumption correct?

Can you stop this flow by closing valves and using pump as compressor?

With the air valves closed and water flowing 3 feet deep in siphon, will the siphon self-prime until it runs full?

What are the main objections to lowering the siphon tubes 3 feet into the dirt of the levee and eliminating sea wall and footings?

What would be the main objection to lowering the siphon tubes 3 feet into the levee and using concrete footings?

It is planned to provide entrance holes in each siphon tube to insert instruments to obtain velocity data. Spot 16 holes to each tube.

In what part of the tube will the greatest turbulence occur?

Which bend will cause the largest loss in hydrostatic head?

If we had flattened and widened the tube at the crown of the levee without decreasing the cross section area, would it affect the volume of flow materially?

If the vacuum is gauged at level 224 feet on each leg of the siphon will the readings be identical if the tube is flowing full?

With five feet differential in pool levels, if the tube is being primed and the water is at level of 224 feet in the upstream level and a vacuum reading is taken at level 224 feet, what point or points on the downstream leg will give identical readings?

If the downstream pool could be lowered as much as 50 feet, at what elevation of downstream pool would the maximum flow through the tube occur?

V. TRASH RACK.

What distinct advantage has a floating boom against the piling of the trash rack over horizontal walings on the pilings?

What factors control the distance the trash rack is placed from the siphon intake?

What chief objection is there to placing trash rack in intake basin and removing trash with a derrick installed on edge of basin?

What is the best method of removing trash from trash rack as installed?

Will trash rack impede flow of water materially at any stage?

Can small trash be shoved under lip of intake to pass through siphon tube?

Do you think vortex action will suck any small trash through the siphon tube?

VI.   OUTLET CHANNEL.

In designing the connecting channel from the discharge basin to the old river channel, what is the maximum mean velocity allowable on a sandy loam bed merging into a light sand as the depth increases? The top layer of soil is buckshot or gumbo.

Left Hand Chute of Little River varies in discharge from a negligible quantity to better than 500 cubic feet per second. How will this influence design of channel?

If the channel is too constricted, will it widen by erosion or deepen and undercut back to the discharge basin?

Granted that the sand in the foundation will compact under a very heavy load per square foot of bearing surface:
   a.   Will this compaction occur largely in the first five feet in depth of foundation?
   b.   Will any of the compaction extend below 20 feet in depth?
   c.   Below 30 feet in depth?

If an overburden of 28 feet of earth and sand is removed from the foundation and the surface of the foundation is not disturbed, can it be correctly assumed that this foundation has already received compaction equivalent to that much weight?

In 1927 a flood was 10 feet in depth over this land. Did that flood add any compaction weight to this foundation?

If all the compaction does occur within the top 20 feet in the foundation:
   a.   Would bearing piles 20 feet long eliminate all the settle- ment due to compaction?
   b.   Piles 30 feet long?

With the foundation sand contained within sheet piling and all the load carried on the sand, will the settlement due to compaction of the sand be less than if no sheet piling was used?

When overburden is removed from a sand foundation, releasing the pressure on same, will the foundation sand expand?

The normal design for seep line is one on six from flow line. In gumbo what change occurs in this seep line when the levee is compacted?

Granted that after compaction of the levee the rear berm is not required by the exposure of the seep line, then what factors make a berm advisable around the discharge end of siphon?

The normal design of a levee is for 10 feet of base for every foot of water against the face of the levee. With approximately 173 feet between the inlet and outlet basins have we sufficient width here where inlet and outlet channels are at elevation 200 feet; and flow line 226 feet?

With the water in lake at 226 feet the inlet basin is submerged:
a. Is the sheet piling under the inlet basin of any value as a cutoff wall?
b. Is the junction of sheet pile and earth a material line of weakness in figuring the shortest seep line under or around the structure and through the levee?
c. In computing the resistances in the lines of seepage, is vertical flow more resistant than horizontal flow in fine sand?

Normally, when water stands against a levee the underground water level rises; its rate of rise depending upon the medium separating the two bodies of water, i.e., filters - seeps - percolates - flows. If a small hole were dug in river bed in front of inlet basin to a depth to hit the water bearing sand, would the increased hydrostatic head of the underground water be dissipated in all directions to raise the general ground water table or would it have an unbalanced flow under the levee?

Behind this levee would you expect a decided lag between the actual rise of the ground water table and the hydrostatic head of the underground flow in the lower strata of coarser sand as shown when this coarse sand is tapped with a well point?

VII. AIR VALVES.

The pipe line to the vacuum pump is 6 inches in diameter. In addition an 8-inch valve is installed on the crown of each siphon tube:
a. Is it necessary to install this 8-inch valve in addition to the 6-inch to break the vacuum at any period of operation?
b. To break the vacuum is there any objection to opening the valve quickly?

If the 6-inch valve is insufficient to break the vacuum at the maximum differential between pools, would it be practicable to utilize excess vacuum suction of air to operate a turbine and electric generator, to furnish electric power to operate lock gates and valves?

Granted that with a decreasing differential between pools, the velocity will decrease until air commences to collect in the crown. In the practicable operation of this siphon will this occur at any stage of operation when the lake level is above elevation 207 feet?
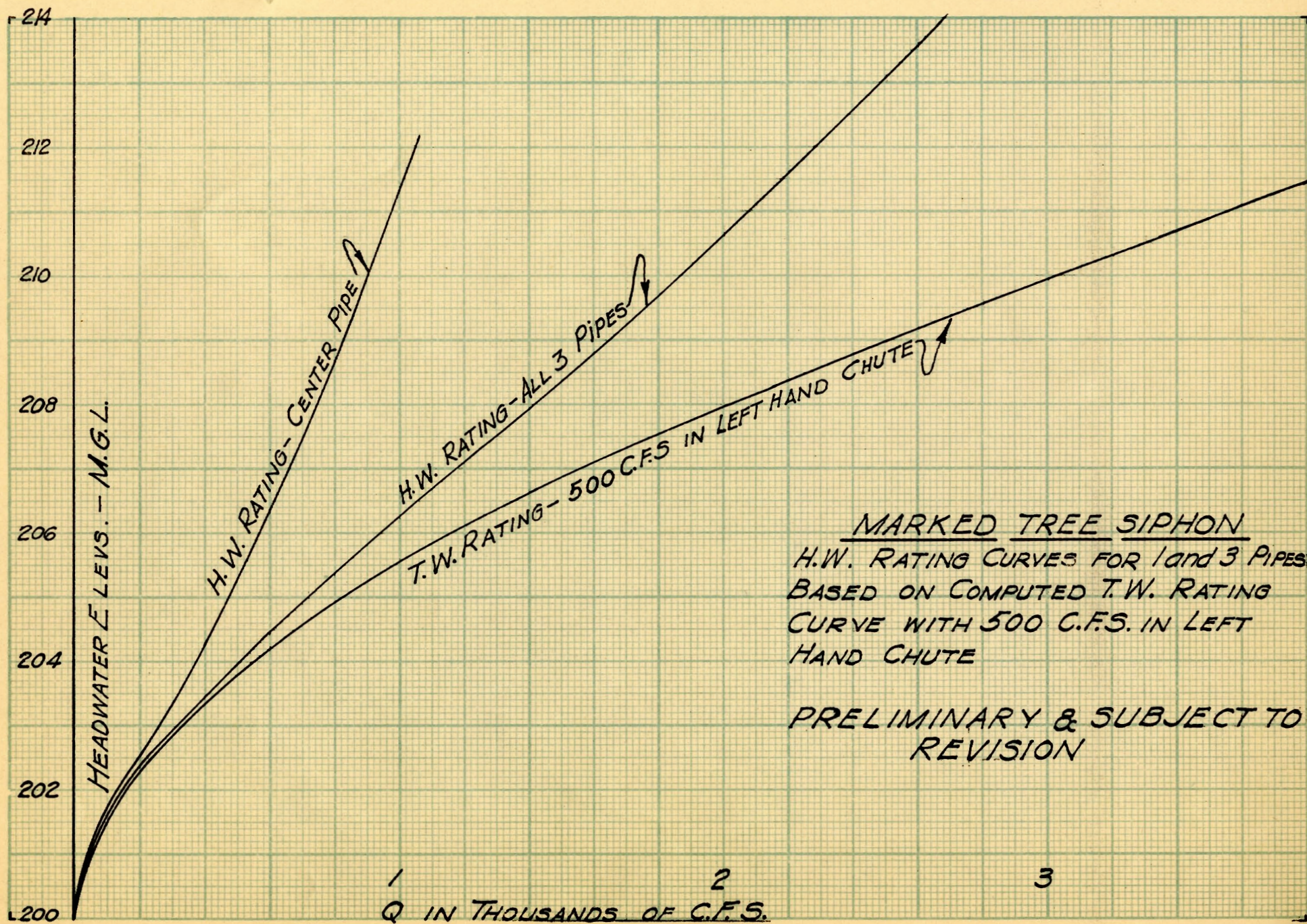
MARKED TREE SIPHON
H.W. RATING CURVES FOR 1 AND 3 PIPES
BASED ON COMPUTED T.W. RATING
CURVE WITH 500 C.F.S. IN LEFT
HAND CHUTE

PRELIMINARY & SUBJECT TO
REVISION

Fig. 1

Water line elevation

219
218
217
216
215
214
213
212
211
210
209
208
207
206
205
204
203
202
201
200

1 PIPE

2 PIPES

3 PIPES

T.W. Elevation

MARKED TREE SIPHON

Hydraulic characteristics
of
Three Barrel Siphon

Curve for tailwater is assumed with no backwater
effect on the tailwater from either left hand chute
or floods which represents a very severe and remote
condition which was used for the design of the discharge
basin and channel.

C.F.S.
1000   Q   2000   3000
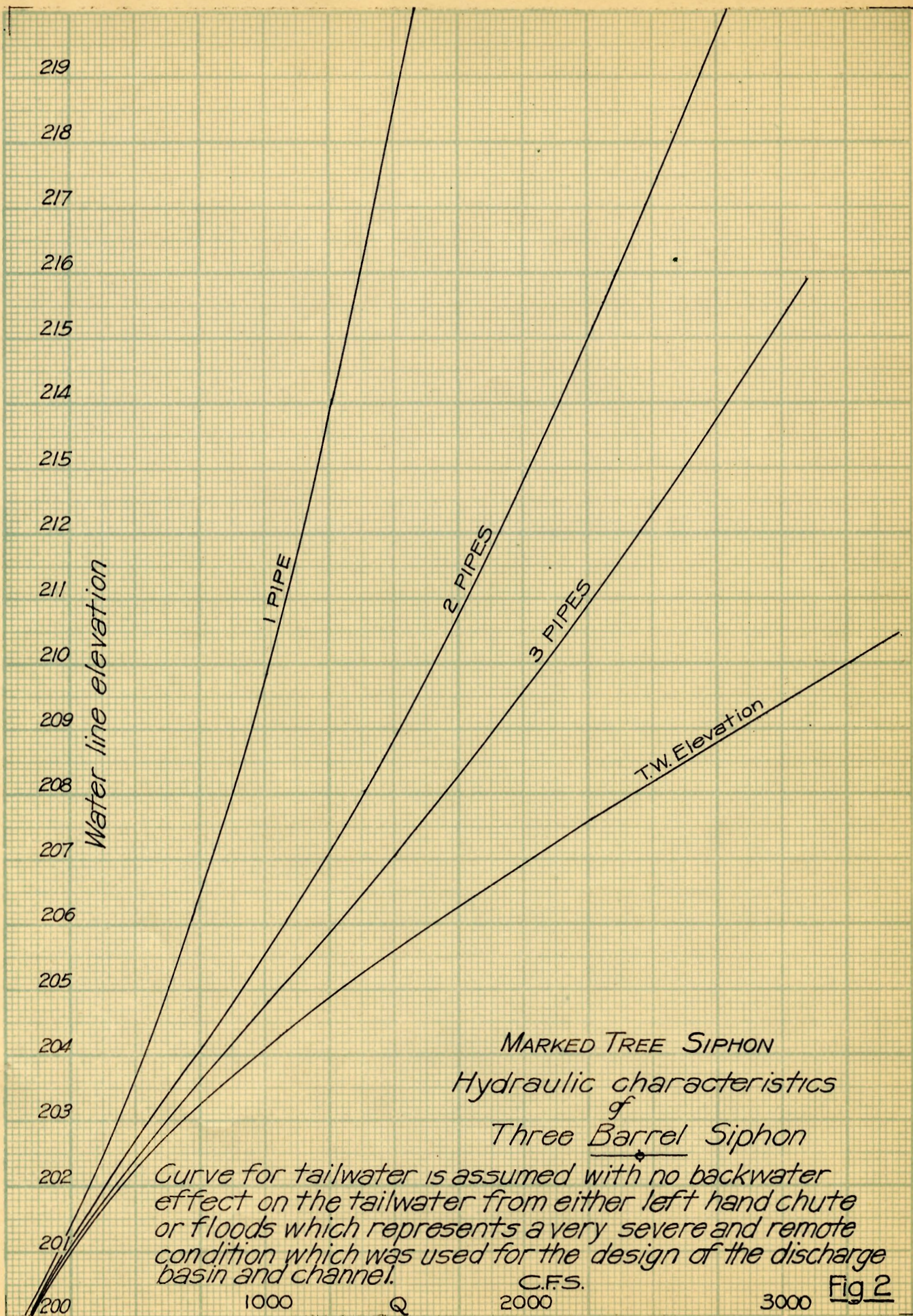
Fig 2

DAILY RECORD SHEET

NOTE: DISCHARGE SCALES ARE FOR
RATING CURVES BASED ON
500 C.F.S. IN LEFT HAND CHUTE

MARKED TREE SIPHON
STAGE HYDROGRAPH FOR CALENDAR YR.
1927
ESTIMATED WET YEAR OF RECORD
AT
UPPER & LOWER ST. FRANCIS LOCK GAGES

Upper Lock Gage

Lower Lock Gage

DISCHARGE TABLE FOR
H.W. RATING CURVE

C.F.S.
2600
2000
1500
1000
800
600
400
200
DISCHARGE SCALE FOR T.W. RATING CURVE
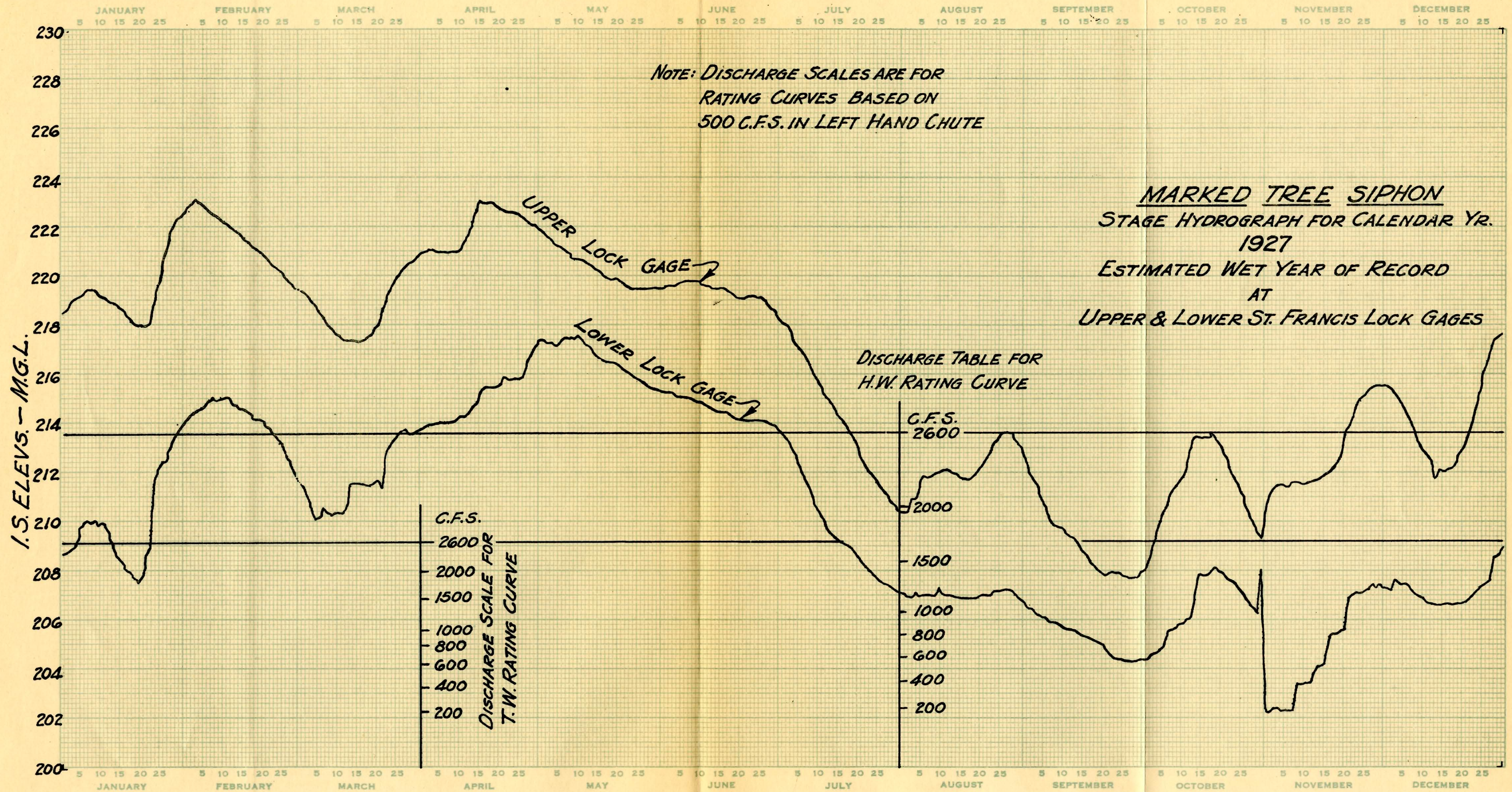
C.F.S.
2600
2000
1500
1000
800
600
400
200
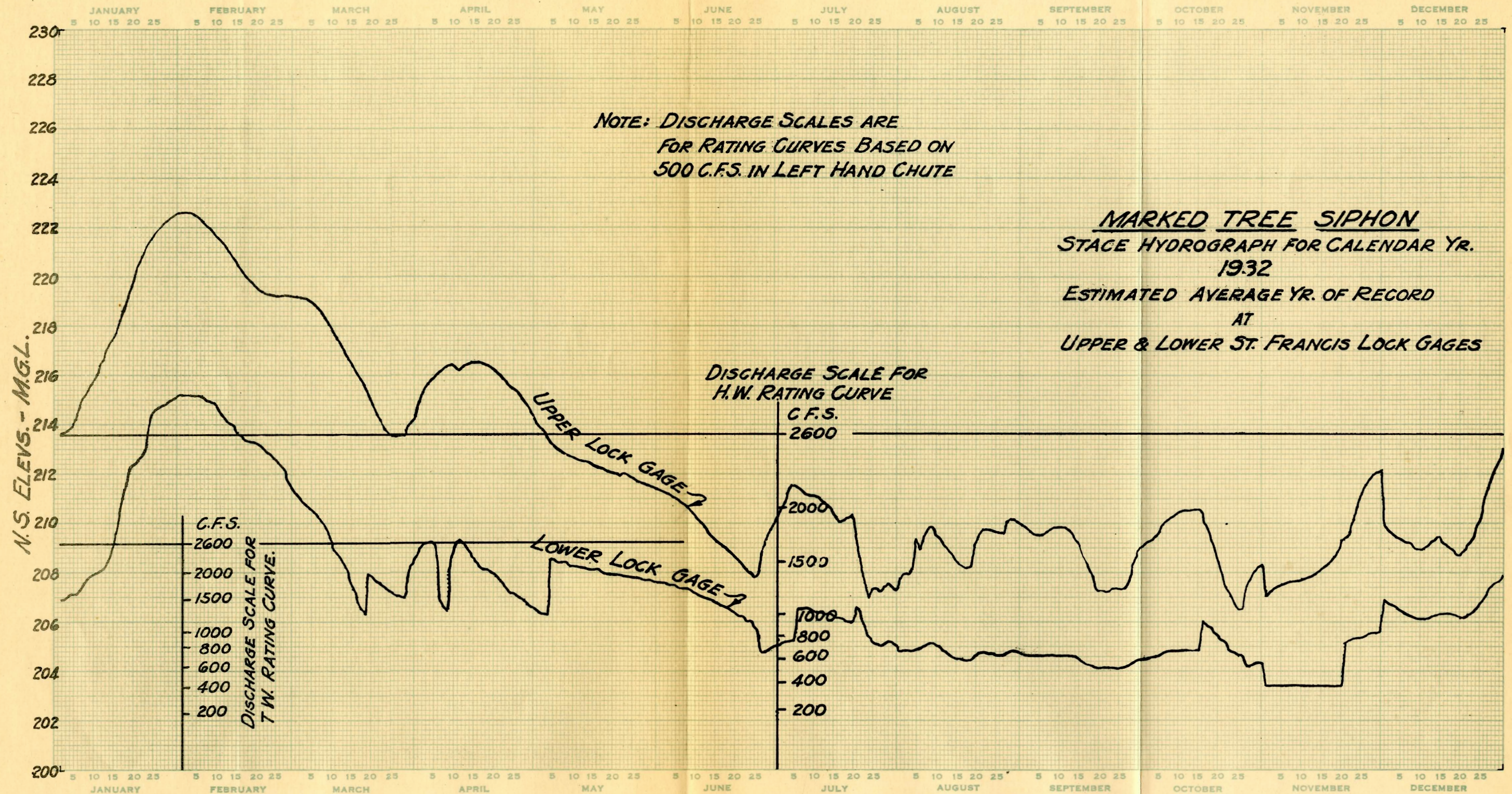
I.S. ELEVS. — M.G.L.

Fig. 3

Fig. 4

NOTE: DISCHARGE SCALES ARE FOR
RATING CURVES BASED ON
500 C.F.S. IN LEFT HAND CHUTE

MARKED TREE SIPHON
STAGE HYDROGRAPH FOR CALENDER YR.
1930
ESTIMATED DRY YR. OF RECORD
AT
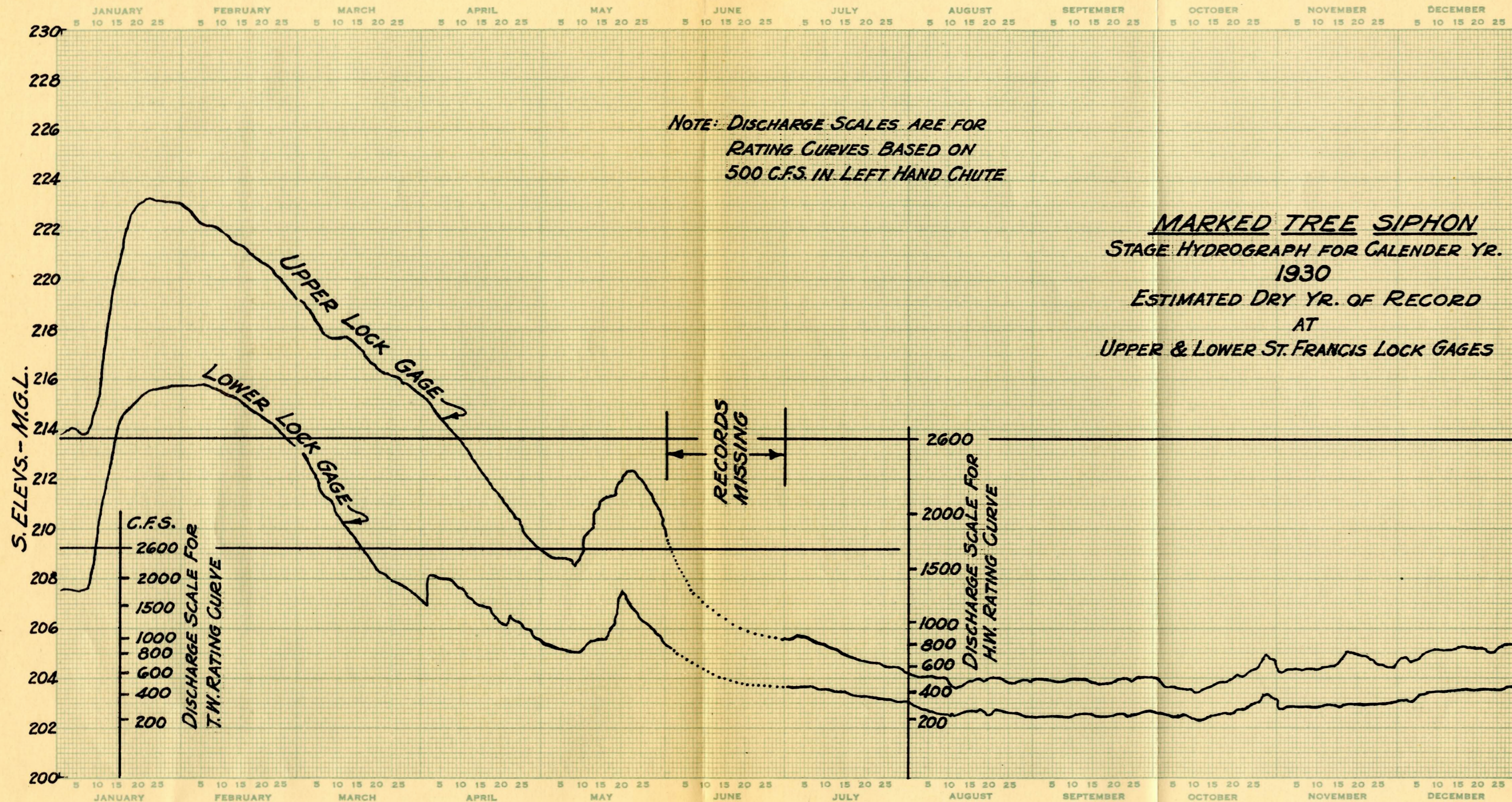UPPER & LOWER ST. FRANCIS LOCK GAGES

Fig. 5

Copy

MARKED TREE SIPHON
Preliminary T.W. Rating Curves as
Represented by Lower Lock Gage
Flow in Left Hand Chute as Parameter
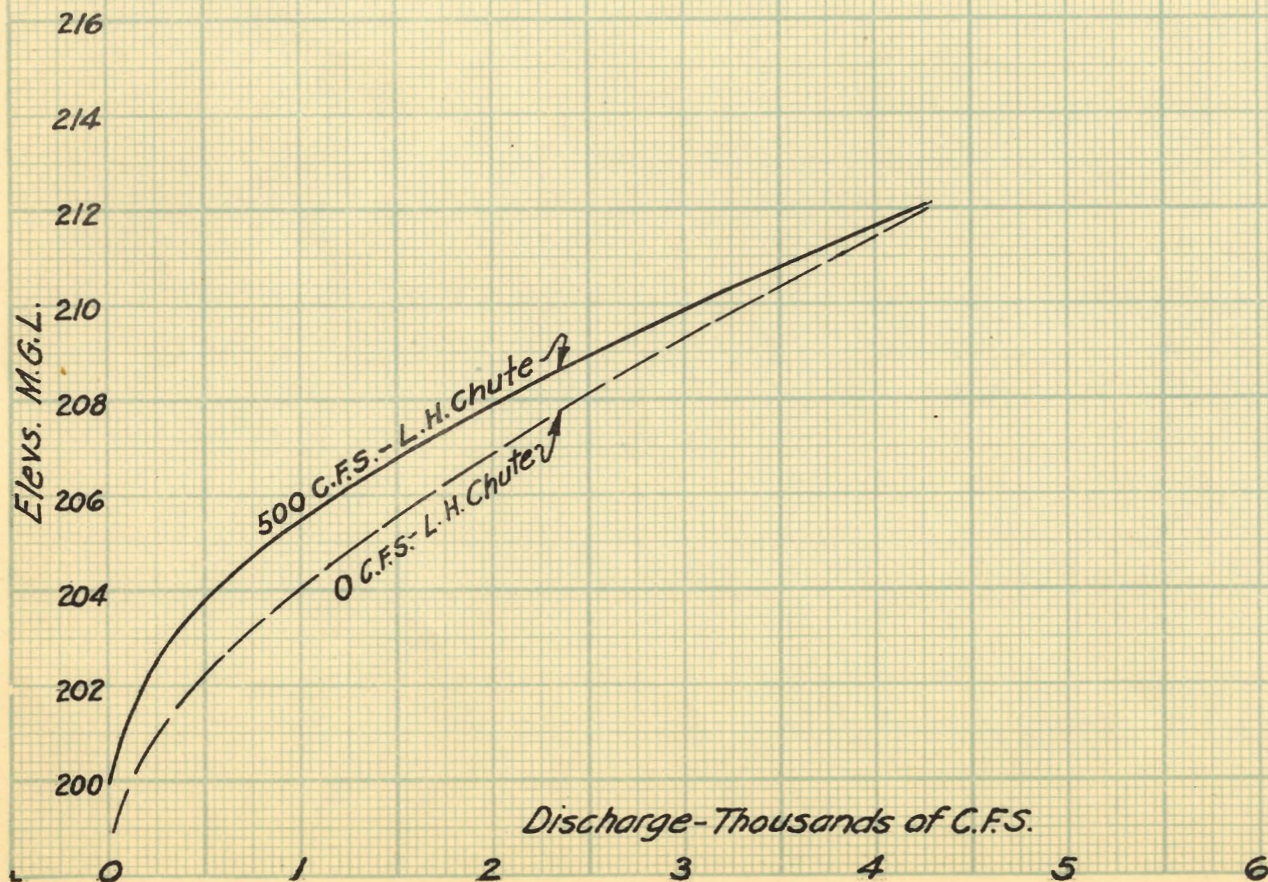
PRELIMINARY & SUBJECT TO CORRECTION

Fig. 6

MARKED TREE SIPHON.
DURATION CURVE OF DAILY TAILWATER
ELEVATIONS AS REPRESENTED BY LOWER
LOCK GAGE, FOR CALENDAR YEAR 1932
WHICH IS ESTIMATED TO PORTRAY
AVERAGE CONDITIONS

PRELIMINARY & SUBJECT TO
REVISION

Fig. 7

PERCENT OF TIME EQUALLED OR EXCEEDED

DATUM IS MEAN GULF LEVEL
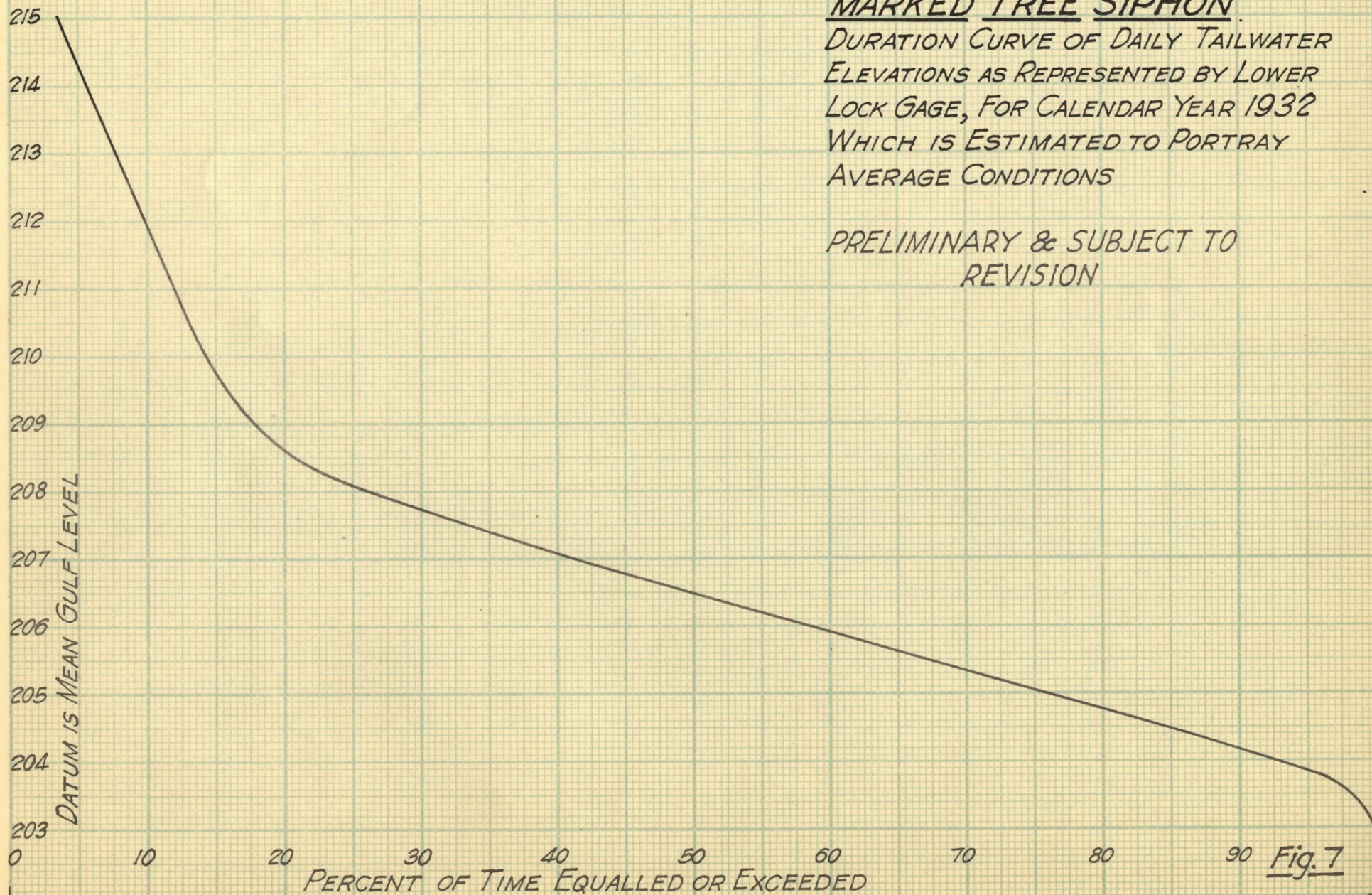
MARKED TREE SIPHON
THEORETICAL CENTERLINE JET
TRAJECTORIES FOR VARIOUS DEPTHS OF
FLOW IN TOP PORTION OF OUTLET LEG

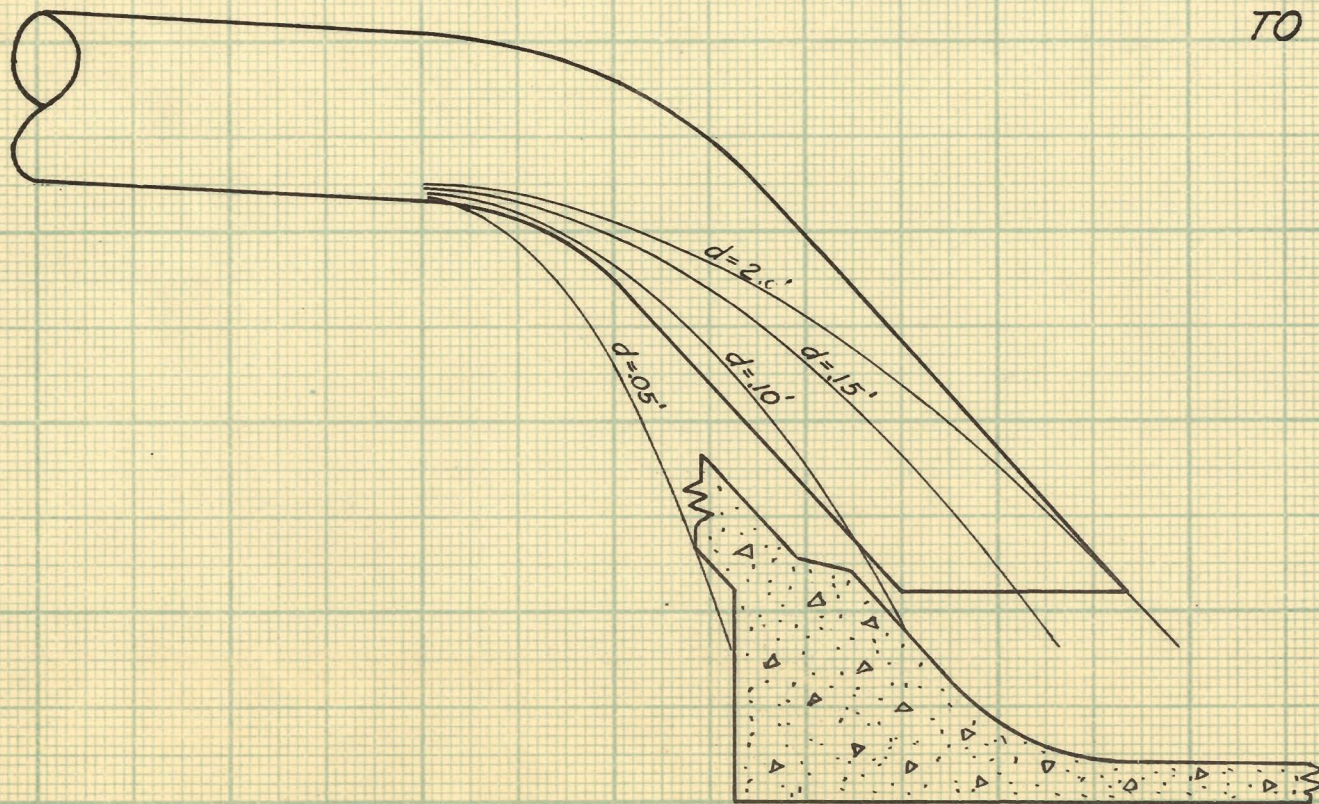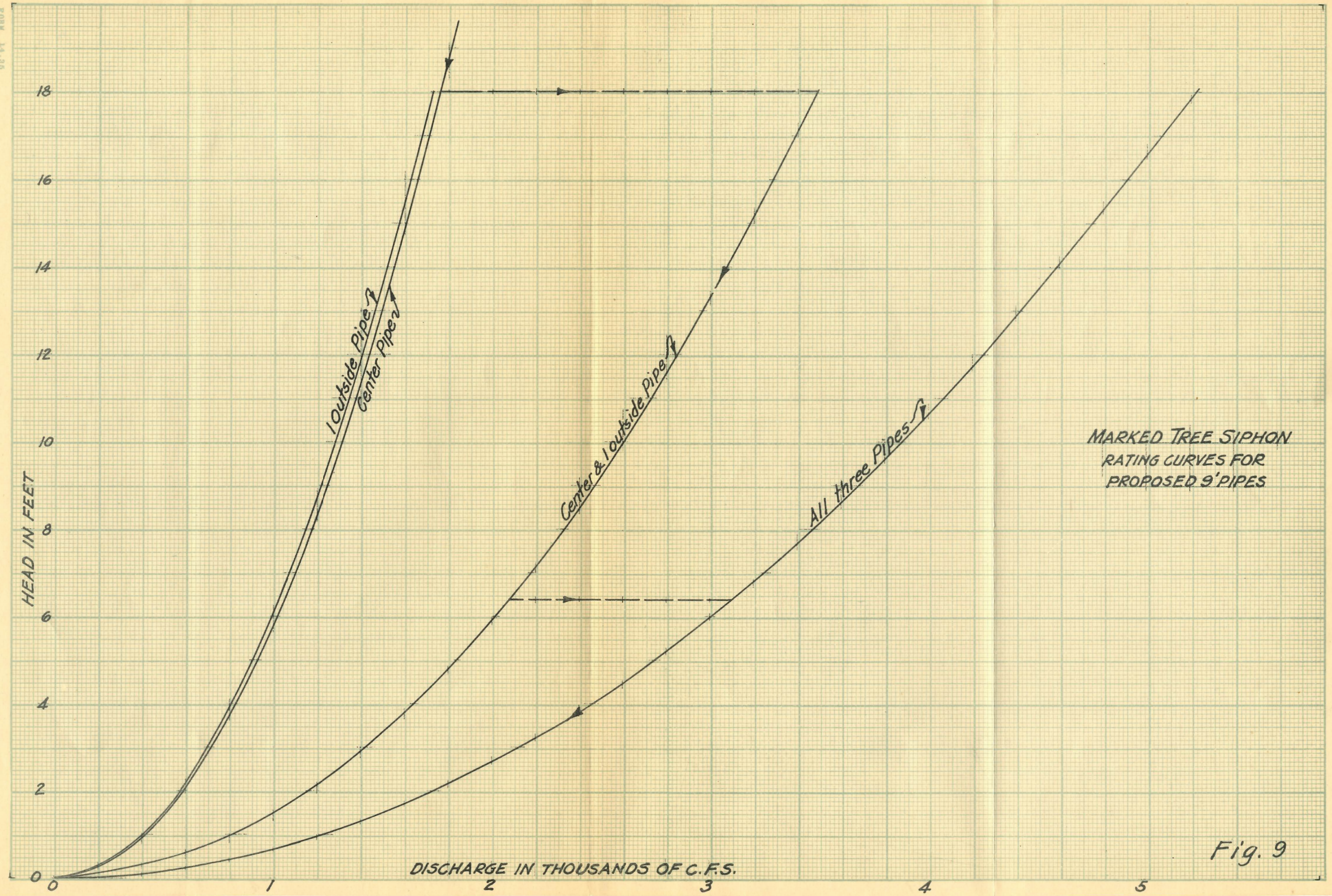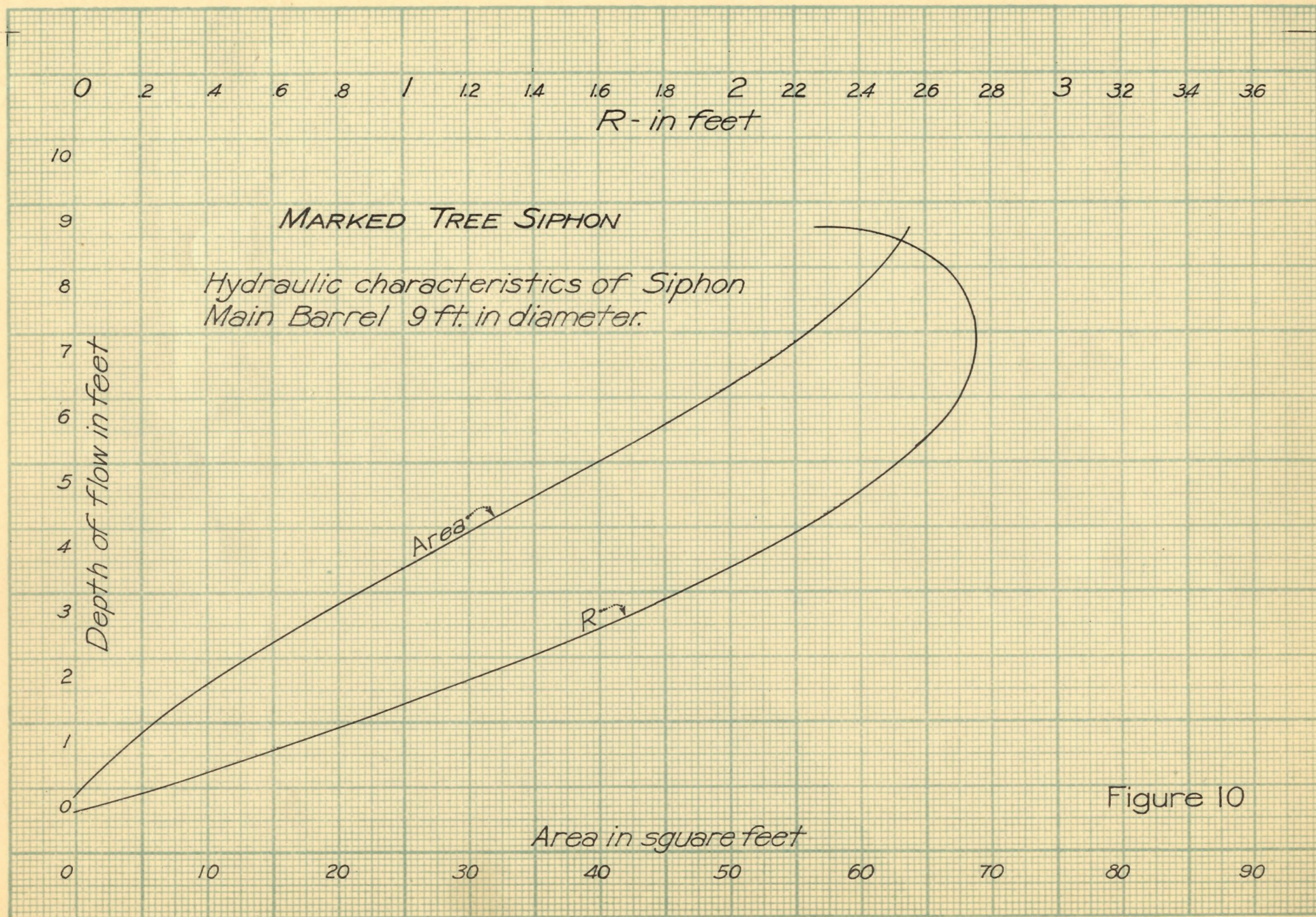PRELIMINARY & SUBJECT
TO REVISION

$d = 2.0'$
$d = 1.5'$
$d = 1.0'$
$d = .05'$

Fig. 8

MARKED TREE SIPHON
RATING CURVES FOR
PROPOSED 9' PIPES

Fig. 9

MARKED TREE SIPHON

Hydraulic characteristics of Siphon
Main Barrel 9 ft. in diameter.

Figure 10

MARKED TREE SIPHON OUTLET BASIN STRESS ANALYSIS

Comp by: R.W.   Checked by: H.C.

SHEAR DIAGRAM
Load Scale: 1"=10,000#

V = 16380# (Max.)

MOMENT DIAGRAM
Load Scale: 1"=100,000'#

M = 14,2000'# (Max.)

Fill

Saturated Soil 60#

Submerged Soil 83#

996#   930#

$H_1 = 24,350$

1926#

Water Surface

$V_1 = 9630$

$H_2 = 4,500$

$V_2 = 13,780$ $V_3 = 23,100$

$H_4 = 9200$

$H_3 = 29,050$

Earth Pressure

Uplift (Hydrostatic)

$V_4 = 30,000$ $V_5 = 16,510$

LOAD DIAGRAM
Load Scale: 1"=1000#
Scale: ⅛"=1'-0"

DESIGN SECTION
Scale: ⅛"=1'-0"

Actual Max. unit stresses in wall slab.
$f_s = 20,200$ #/□" $f_c = 670$ #/□"

¾"φ @ 12" C.C.

Actual max. unit stresses in base slab
$f_s = 20,000$ #/□" $f_c = 670$ #/□"

Symmetrical about center line

2'-0" Slab

1¼"□ @ 7½" c.c. below Elev. 203.0

1¼"□ @ 15 c.c. above El. 203.0

1¼"□ @ 8 c.c. alternate, top & bottom both ways

MOMENT DIAGRAM
Load scale: 1"=100,000'#

+45000'#

M = -142000'# (Max)

SHEAR DIAGRAM
Load Scale: 1"= 4000#

V = 5210#
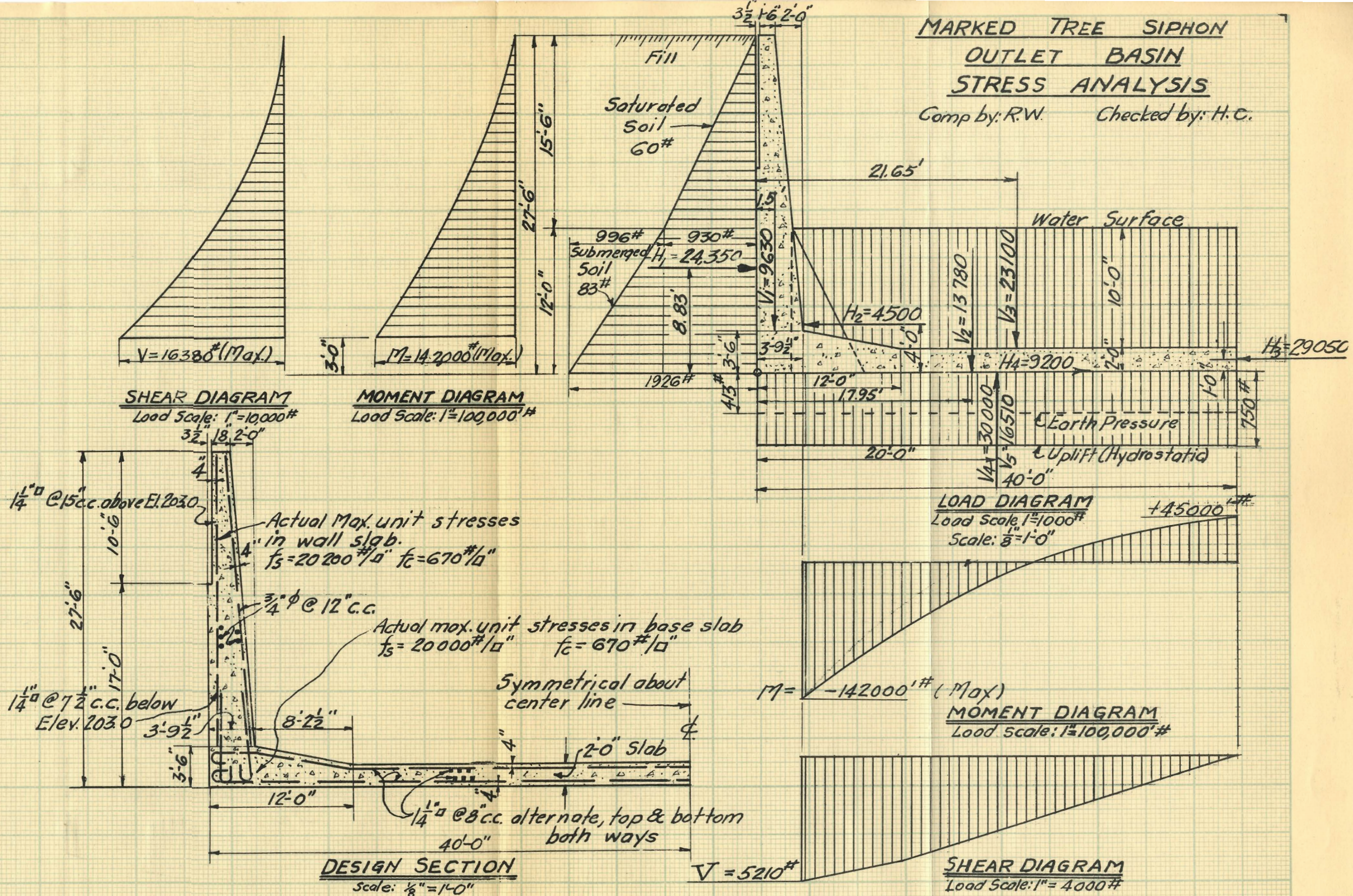
Fig. 11

SUBJECT _St. Francis River Project   Marked Tree Siphon_

COMPUTATION _Concrete Design – Outlet Basin_   NO. ____

COMPUTED BY _H. Cole_ ____ CHECKED BY ____   DATE _9-22-38_

OUTLET BASIN

1. <u>Wall Steel</u> $A_s = \frac{1}{4}'' \square @ 7\frac{1}{2}'' c.c. = 2.50 \square''$ $d = 42'' - 4'' = 38''$

$$M = -142000 \qquad V = 16380^{\#}$$

Steel Tension, $f_s = \dfrac{142000 \times 12}{0.89 \times 2.5 \times 38} = 20200^{\#}\!/sq.inch$

Compr. Concrete, $f_c = \dfrac{2 \times 142000 \times 12}{0.33 \times 0.89 \times 12 \times 38 \times 38} = 670^{\#}\!/sq.inch$

Shear, $v = \dfrac{16380}{0.89 \times 6.25 \times 38} = 41^{\#}\!/sq.inch$

Bond, $u = \dfrac{16380}{0.89 \times 6.25 \times 38} = 77^{\#}\!/sq.inch$

---

2. <u>Base Slab Steel</u> $A_s = \frac{1}{4}'' \square @ 8'' c.c. = 2.34 \square''$ $d = 24'' - 4'' = 20''$

Design   24" Base Slab – Span 80'-0" (Assume)

$$W = 2 \times (150 - 62.5) = 175^{\#} (2 \text{ ways, } 87.5 \text{ Each way})$$

$$M = \frac{1}{8} \times 87.5 \times \overline{80}^2 = 70000'^{\#}$$

<u>Reinforcing at
Bottom of Slab</u>

$$f_s = \frac{70000}{0.89 \times 2.34 \times 20} = 20000^{\#}\!/sq.inch$$

<u>Reinforcing at Top   (Center of Slab)</u>

$$M_R = A_s f_s jd = \frac{2.34 \times 20000 \times 0.89 \times 20}{12} = 70000'^{\#}$$

Net upward pressure $M = 70000 + 70000 = 140000'^{\#}$

Net upward hydrostatic pressure $\dfrac{Wl^2}{8} = 140000$

$$W = \frac{140000 \times 8}{80 \times 80} = 175^{\#} load \ (net)$$

Maximum pressure head   24" Slab = 300

$$\underline{W \quad = 175 \, (Net)}$$

475 (Unwatered Condition)

$$h = \frac{475}{62.5} = 7.6 \ feet$$

FIG. 12

SUBJECT _St. Francis River Project   Marked Tree Siphon_

COMPUTATION _Outlet Basin_   NO. ____

COMPUTED BY _R. Wedemeyer_   CHECKED BY _H. Cole_   DATE _9-16-38_

Outlet Basin

El. 215.5

El. 199.0

16'-3"

9'-10"

125 ᵏ

169 ᵏ

$R_2 = 169^k$

Assume
R = 0

20'-0"

35'-0"

$R_1 = 60^k$

9'-0" Dia. Pipe

Scale: ¹/₁₀" = 1'-0"

FIG. 13

SUBJECT. _St. Francis River Project   Marked Tree Siphon_

COMPUTATION _Outlet Basin_   NO. _____

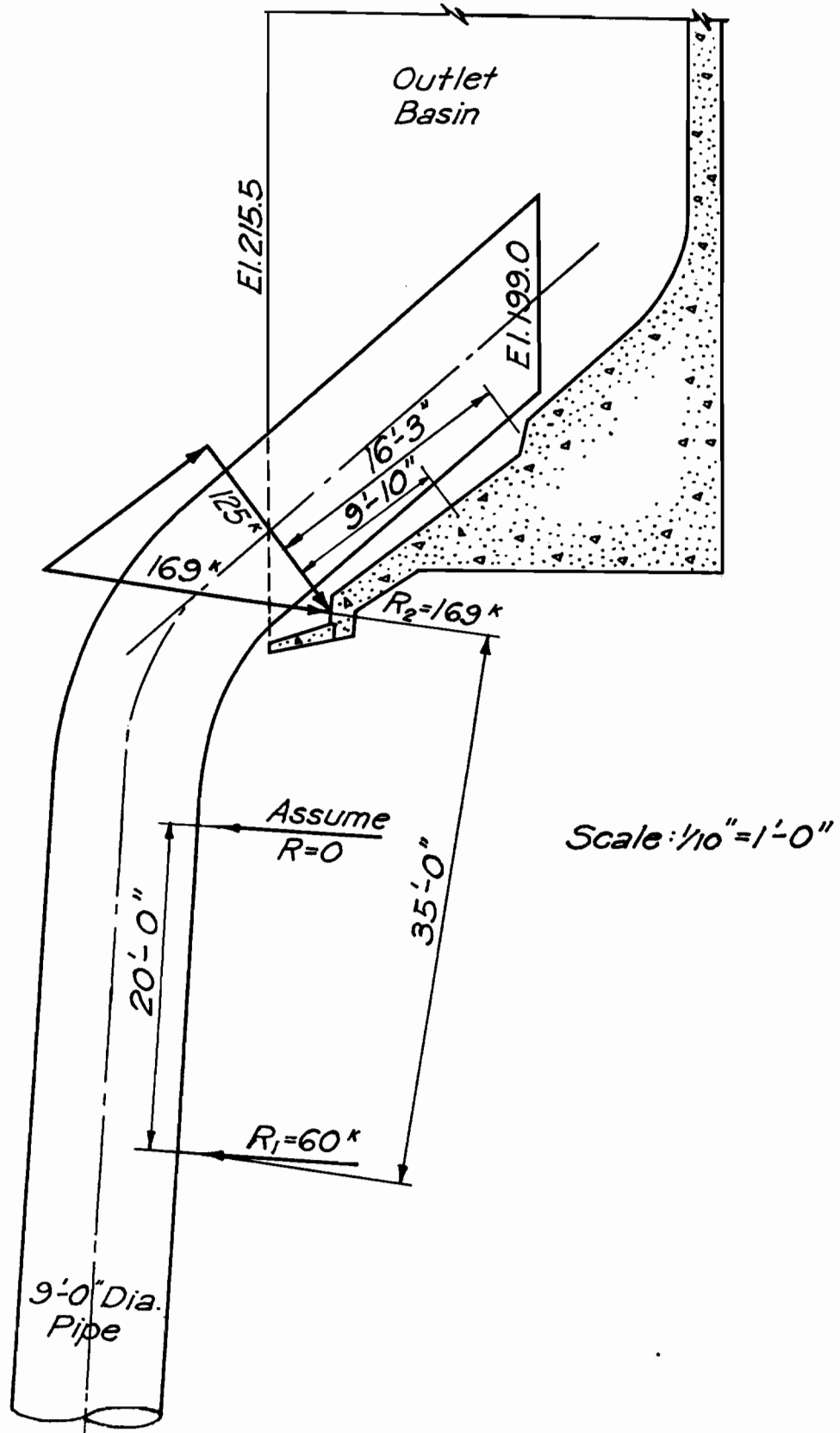COMPUTED BY _R. Wedemeyer_   CHECKED BY _H. Cole_   DATE _9-16-38_

Revised 9-22-38

El. 215.5

6"

3/4"∅ @ 12"c.c.
1/2"∅ Spacers

1'-10 5/8"

8'-7 3/4"

5'-0"

10"

1'-6

1'-6

3 1/2"

12'-3 1/2"

7'-0"

1"□ @ 8 c.c.

3/4"∅ @ 12" c.c.

4"

El. 205.0

1'-1 1/4"   1'-7 1/4"   1'-11 1/2"

4'-8"

OUTLET BASIN
Scale: 1/2"=1'-0"

FIG. 14

SUBJECT _St. Francis River Project   Marked Tree Siphon_
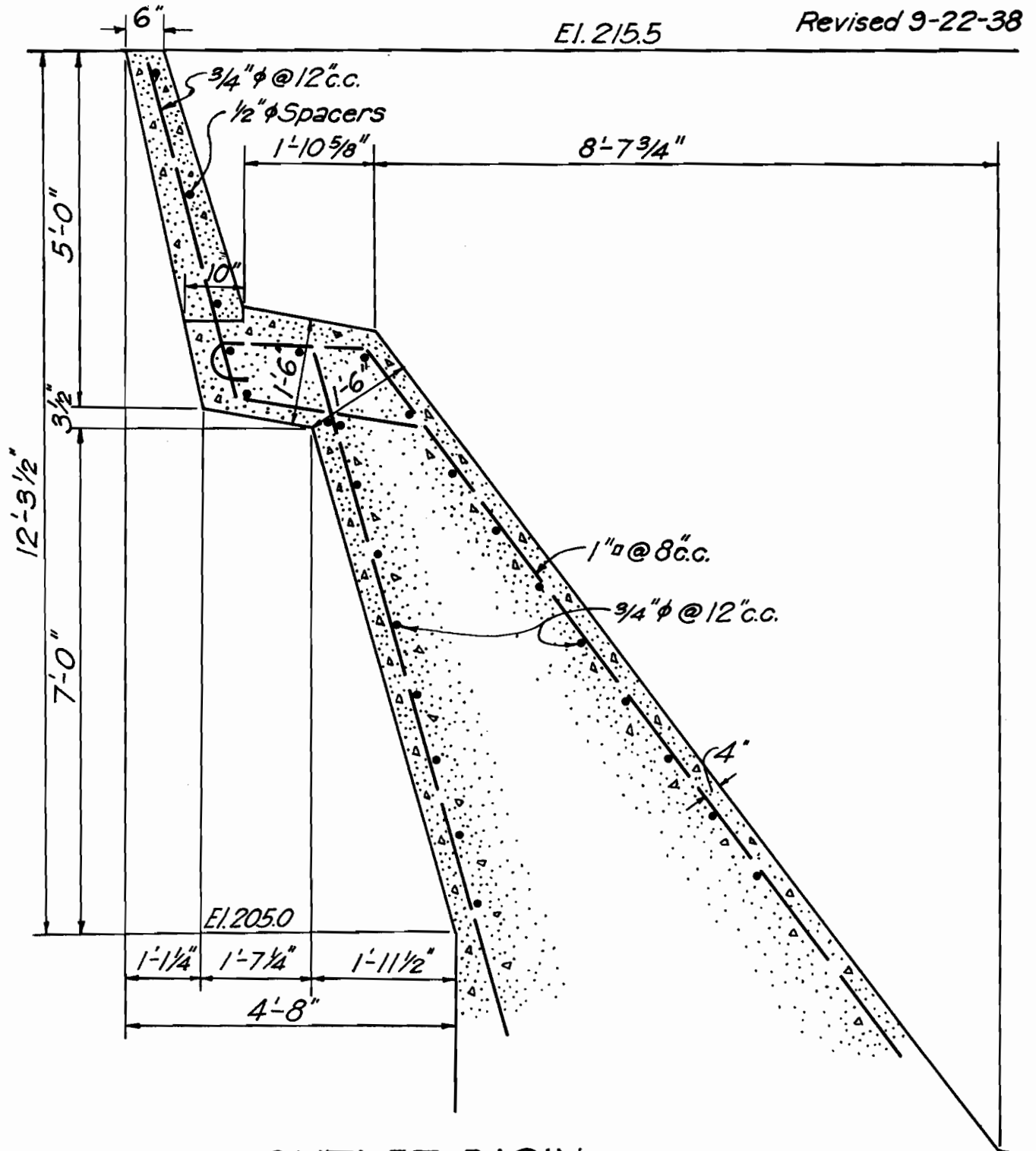
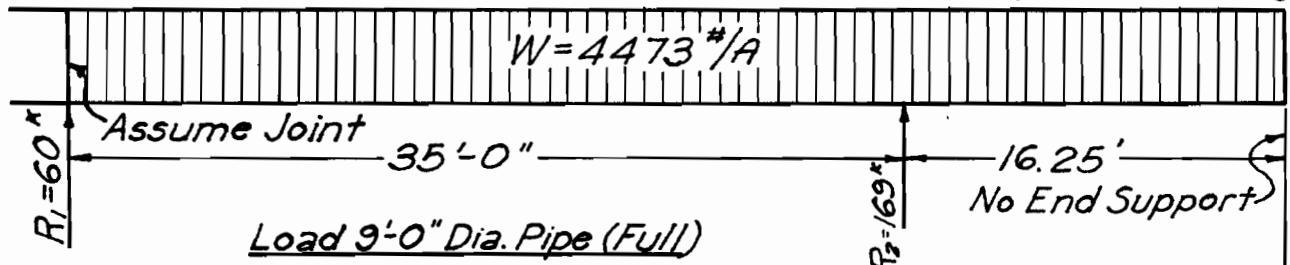COMPUTATION _Outlet Basin_   NO._____

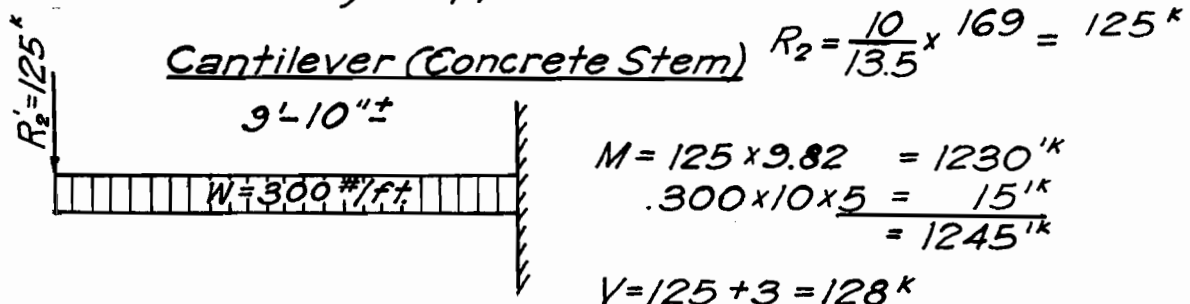COMPUTED BY _R. Wedemeyer_   CHECKED BY _H. Cole_   DATE _9-16-38_
                                                         Revised 9-22-38

$$W = 4473 \,^{\#}/A$$

Assume Joint

$R_1 = 60^k$ ←—— 35'-0" ——→ $R_2 = 169^k$ ←— 16.25' —→   _No End Support_

Load 9'-0" Dia. Pipe (Full)

W   Steel ³/₈ Plate $3.14 \times 9.0 \times 15.30 = 430^{\#}$
     Steel Struct. Details = +10% = $\underline{43^{\#}}$
                                    $473^{\#}$

     Water $0.7854 \times 9^2 \times 62.5 =$

                        Steel = $473^{\#}$
                        Water = $\underline{4000^{\#}}$
                        Total W = $4473^{\#}$

$$R_2 = \frac{4473 \times 51.25 \times 25.62}{35} = 169000^{\#}$$

$$R_1 = 229000 - 169000 = 60000^{\#}$$

Design Support for $169000^{\#}$

$R_2' = 125^k$

Cantilever (Concrete Stem) $R_2 = \frac{10}{13.5} \times 169 = 125^k$

←—— 9'-10"± ——→

$W = 300 \,^{\#}/ft.$

$$M = 125 \times 9.82 = 1230^{'k}$$
$$.300 \times 10 \times \underline{5} = \underline{15^{'k}}$$
$$= 1245^{'k}$$

$$V = 125 + 3 = 128^k$$

For Shear = $d = \dfrac{128000}{\frac{7}{8} \times 120 \times 90} = 13.6"$ _Minimum 18" deep. (top)_

For Bending = $d = \sqrt{\dfrac{1245 \times 12}{197 \times 216}} = 18.7"$

                                    Steel $1"^{\square}$ @ 9 c.c.
$M_{Top} = 125 \times 1.5 = 187.5^{'\#}$ $A_s = 1.50^{"\square}$

$$f_s = \frac{187.5 \times 12}{.89 \times 1.5 \times 14} = 12,000 \,^{\#}/_{\square}" \; O.K.$$

$$\mathcal{w} = \frac{125,000}{.89 \times 6.0 \times 14} = 167 \,^{\#}/_{\square}" - 1, K \text{ for bond}$$
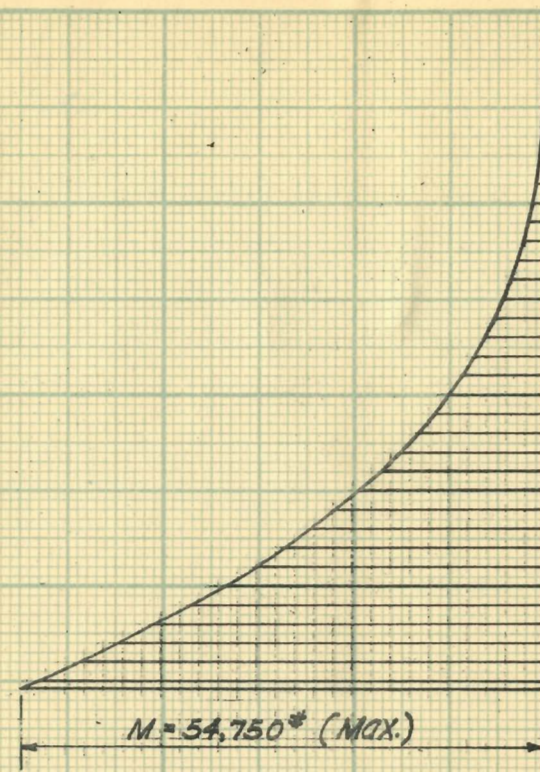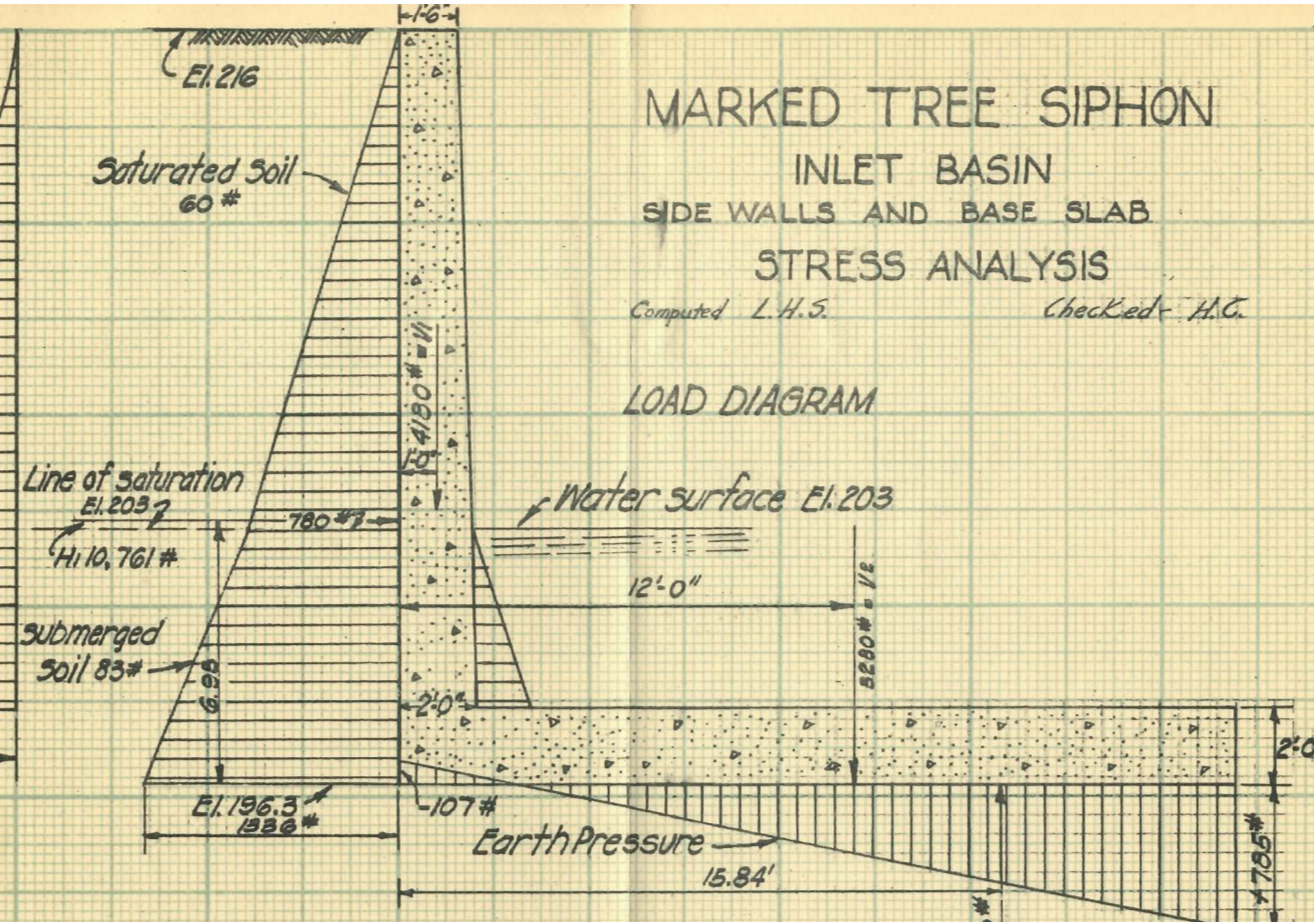
$$f_s \text{(bottom)} = \frac{1245 \times 12}{.89 \times 1.5 \times 46} = \underline{24,200} \text{ (O.K. + 25\% for Emergency)}$$
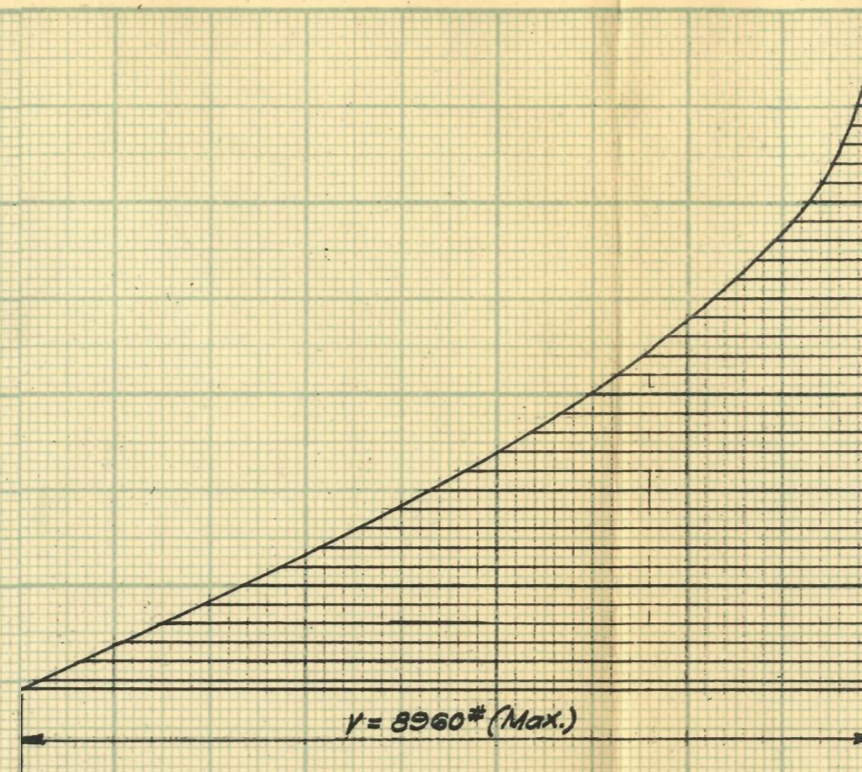
FIG. 15

MARKED TREE SIPHON
INLET BASIN
SIDE WALLS AND BASE SLAB
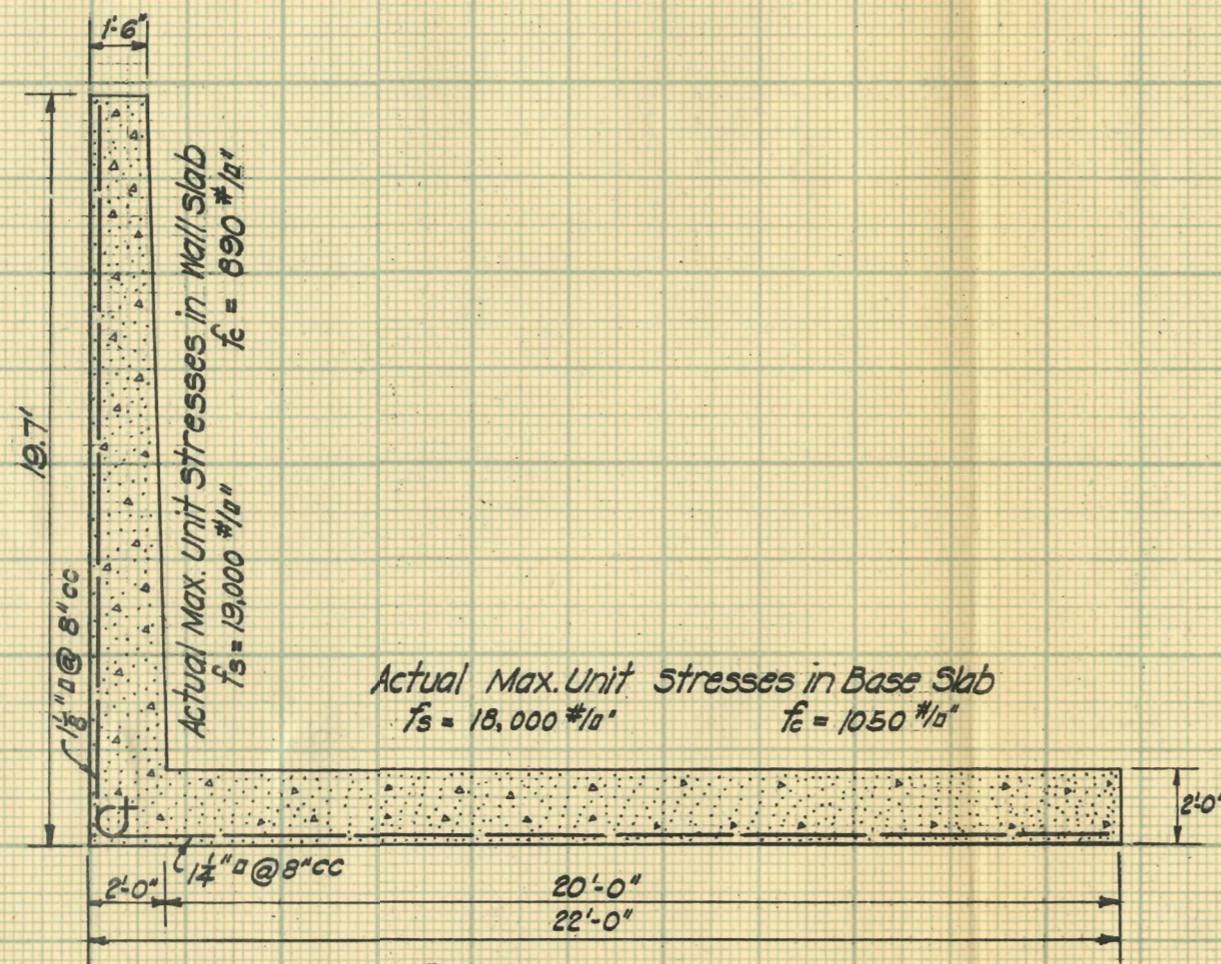STRESS ANALYSIS
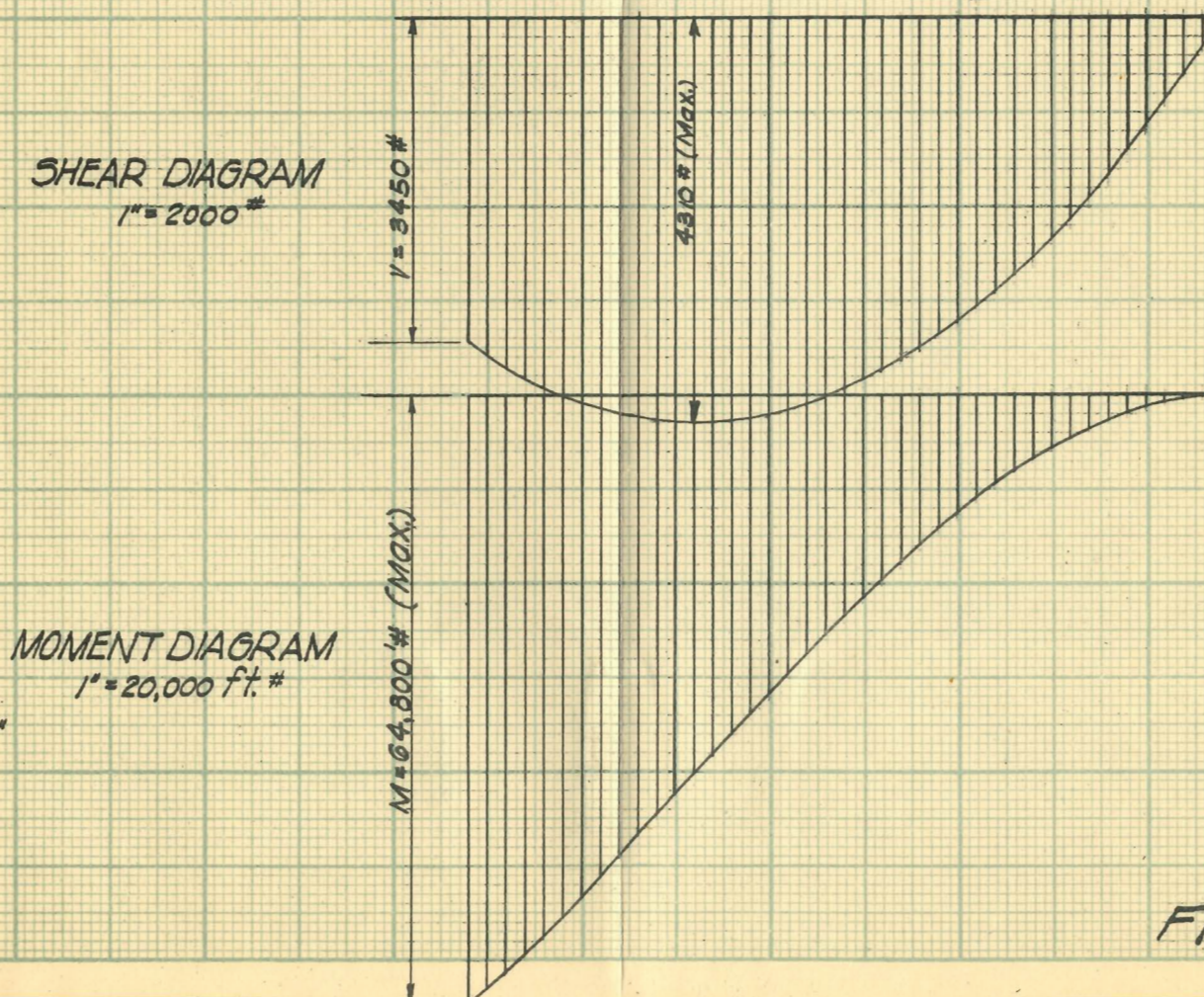Computed L.H.S.   Checked H.C.

LOAD DIAGRAM

El. 216
Saturated Soil 60#
Line of saturation El.203
H 10,761#
submerged Soil 83#
780 #
El. 196.3 1838
-107#
Earth Pressure
15.84'
Water surface El.203
12'-0"
5280 # 12
2'-0"
4180# 11
6.96
2'-0"
7760#
7705#

MOMENT DIAGRAM
1" = 20,000 ft. #
M = 54,750# (Max.)

SHEAR DIAGRAM
1" = 2000#
V = 8960# (Max.)

SHEAR DIAGRAM
1" = 2000#
V = 3450#
4810# (Max.)

MOMENT DIAGRAM
1" = 20,000 ft. #
M = 64,800# (MAX.)

DESIGN SECTION
1" = 5'

1'-6"
19.7'
1¼"ø @ 8"c.c.

Actual Max. Unit Stresses in Wall Slab
fs = 19,000 #/□"   fc = 890 #/□"

Actual Max. Unit Stresses in Base Slab
fs = 18,000 #/□"   fc = 1050 #/□"

1¼"ø @ 8"c.c.
2'-0"
2'-0"   20'-0"
22'-0"

Fig. 16

SUBJECT _St. Francis River Project   Marked Tree Siphon_

COMPUTATION _Concrete Design_ _____ NO. _____

COMPUTED BY _H. Cole_ ____ CHECKED BY _____ DATE _9-22-38_

INLET BASIN

1.   _Stresses at bottom of wall_ $\dfrac{24" Slab}{1\frac{1}{8}° @ 8" c.c.}$

$$d = 24" - 4" = 20", \quad A_s = 1.90^{\square "}$$
$$z_o = 5.31"$$

$$M = -54,750'^{\#} (Max)$$
$$V = 8972^{\#} (Max)$$

Tension-Steel $f_s = \dfrac{54,750 \times 12}{.89 \times 1.90 \times 20}$ $=$ $\underline{19,500^{\#/\square "}}$

Comp. Concrete $f_c = \dfrac{2 \times 54,750 \times 12}{.89 \times .33 \times 12 \times \overline{20}^2}$ $=$ $\underline{940^{\#/\square "}}$

Shear, $V = \dfrac{8972}{.89 \times 12 \times 20}$ $=$ $\underline{42^{\#/\square "}}$

Bond, $u = \dfrac{V}{j z_o d} = \dfrac{8972}{.89 \times 5.31 \times 20}$ $=$ $\underline{94^{\#/\square "}}$

---

2.   _Stresses of Cantilever Base Slab._   _(24" slab)_

$$A_s = 1\frac{1}{4}^{\square} @ 8" c.c. = 2.34"$$
$$M = -64,800'^{\#} (Max)$$
$$V = 4,300 \ (Max) \quad z_o \quad " \quad " \quad = 7.50"$$

Tension Steel, $f_s = \dfrac{64,800 \times 12}{.89 \times 2.34 \times 20}$ $=$ $\underline{18,600^{\#/\square "}}$

Comp. Concrete, $f_c = \dfrac{2 \times 64,800 \times 12}{.89 \times .33 \times 12 \times \overline{20}^2} = \underline{1100^{\#/\square "}}$

Shear, $V = \dfrac{4300}{.89 \times 12 \times 20}$ $= \underline{20^{\#/\square "}}$

Bond., $u = \dfrac{4300}{.89 \times 7.5 \times 20}$ $= \underline{32^{\#/\square "}}$

NOTE
   Moments taken at edge of slab.

FIG. 17

SUBJECT _St. Francis River Project   Marked Tree Siphon_

COMPUTATION _Inlet Basin_   NO. _____

COMPUTED BY _R. Wedemeyer_   CHECKED BY _H. Cole_   DATE _9-16-38_
Revised _9-22-38_

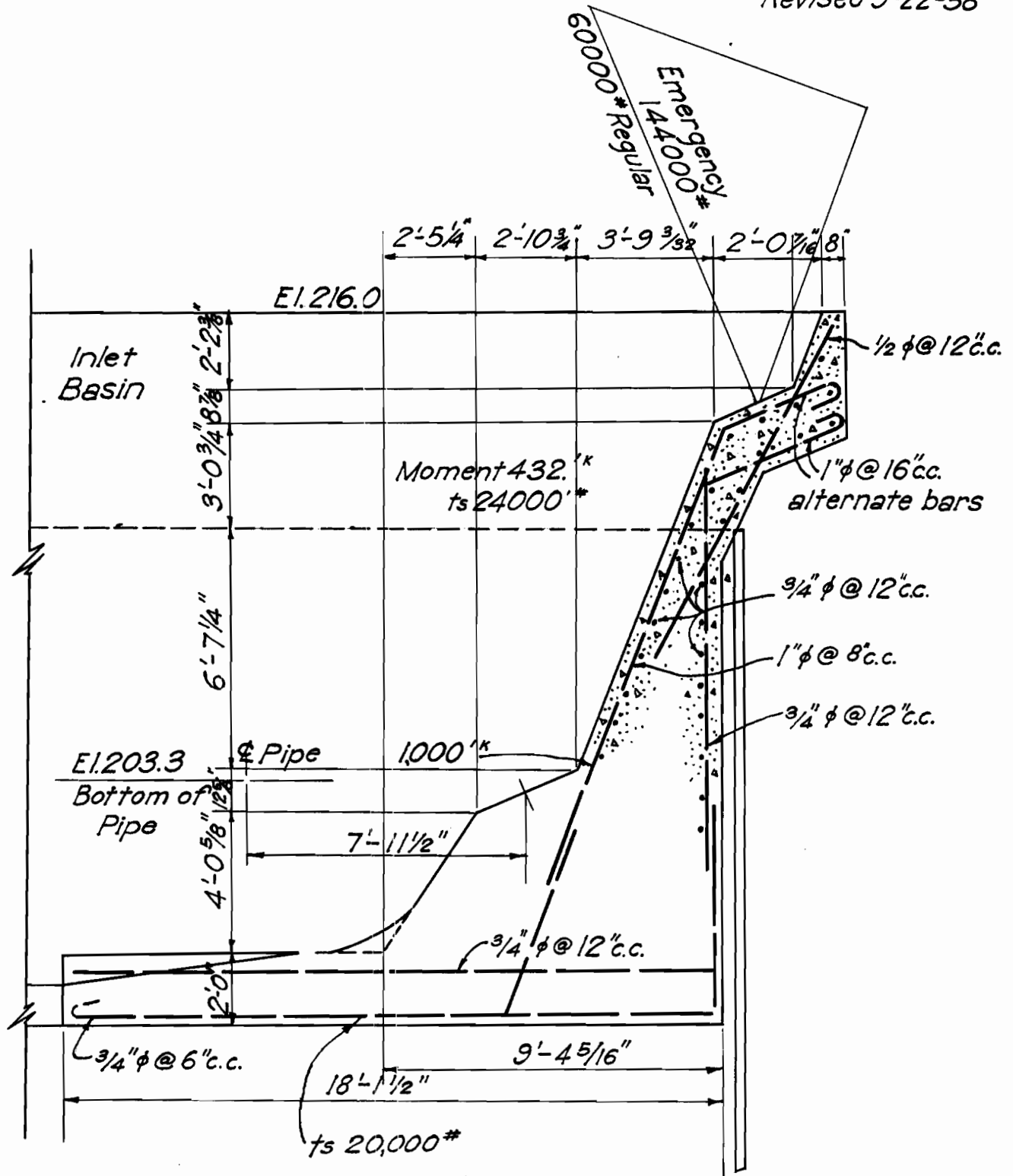FIG. 18

SUBJECT _St. Francis River Project Marked Tree Siphon_

COMPUTATION _Inlet Basin Case I (Emergency)_ NO.

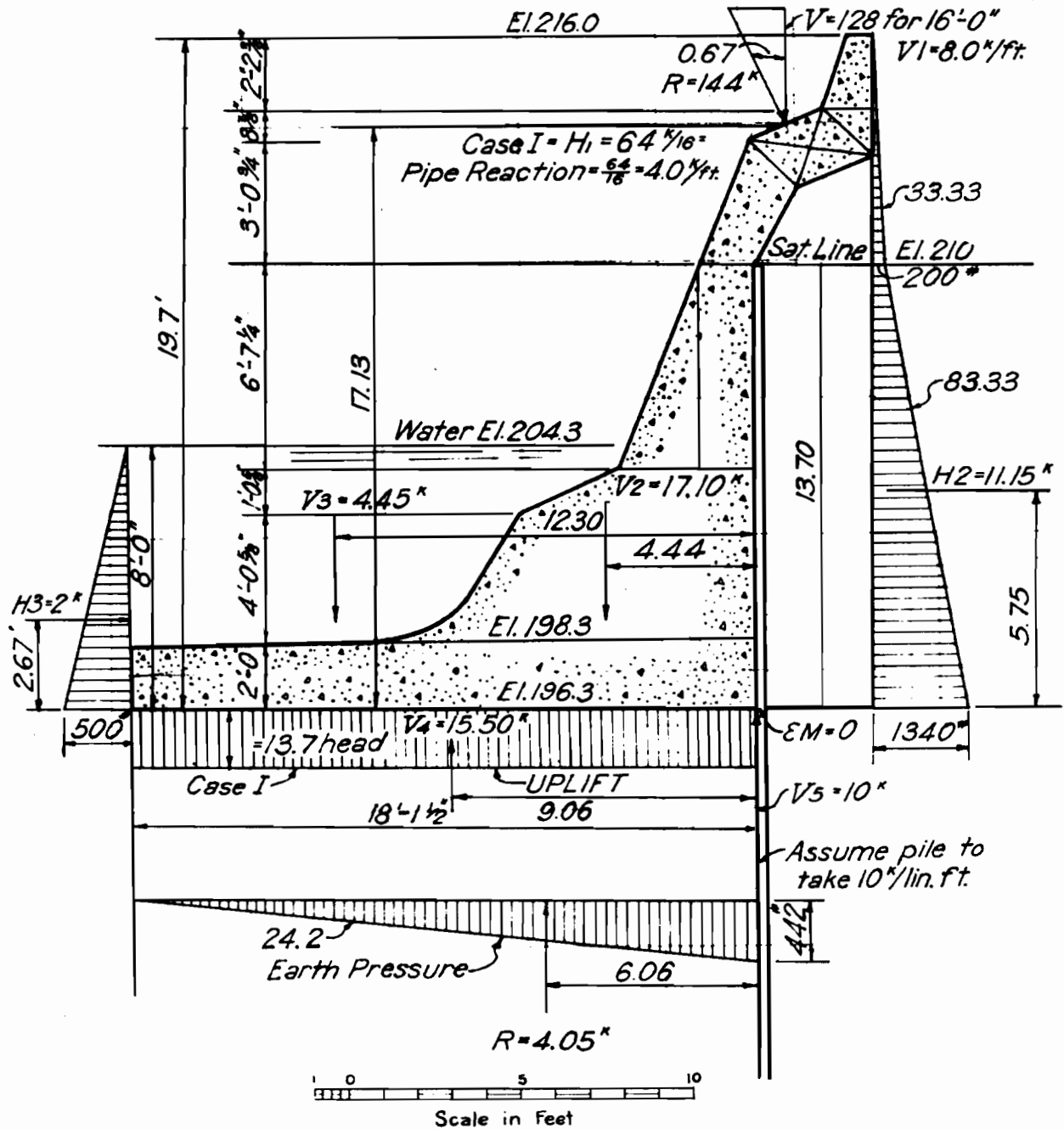COMPUTED BY _H. Cole_ CHECKED BY DATE _9-16-38_

Revised 9-23-38

El. 216.0 $V = 128$ for 16'-0"

0.67 $V1 = 8.0^k/ft$.

$R = 144^k$

33.33

Case I - $H_1 = 64\, ^5/_{16}$ =

Pipe Reaction = $\frac{64}{16} = 4.0^k/ft$.

Sat. Line   El. 210

200 #

83.33

Water El. 204.3

$V2 = 17.10^k$

$V3 = 4.45^k$ 12.30

4.44 $H2 = 11.15^k$

13.70

El. 198.3

5.75

$H3 = 2^k$ El. 196.3

$\Sigma M = 0$ 1340 #

500   $V4 = 15.50^k$

= 13.7 head

Case I   UPLIFT $V5 = 10^k$

18'-1½'   9.06

Assume pile to take 10"/lin. ft.

24.2

Earth Pressure   6.06

$R = 4.05^k$

Scale in Feet

FIG. 19